NOMINATION FOR NATIONAL ENGINEERING LANDMARK

LAKE BURLEY GRIFFIN SCHEME IN CANBERRA A.C.T.

Photo courtesy National Capital Authority

February 2001

22 February 2001

Commemorative Plaque Sub-Committee
The Institution of Engineers, Australia
Engineering House
11 National Circuit
BARTON ACT 2600

The Institution of Engineers, Australia

CANBERRA DIVISION

The following work is nominated for a:

National Engineering Landmark

Name of work:

Lake Burley Griffin Scheme, consisting of Lake impounded by Scrivener Dam, and Commonwealth and Kings Ave. bridges.

Location, including address and map grid reference:

Canberra, A.C.T. – 1:25000 CMA Map 8727-111-N,
central point reference is 35° 18' (lat) 149°07 (long)

Owner:

Commonwealth of Australia – administered by the National Capital Authority (10-12 Brisbane Ave. Barton, ACT 2600)

The owner has been advised of the nomination of the work (refer to Section 9)

The Authority's officers have assisted with information for this submission, in anticipation of formal response by Authority.

Access to site:

Foreshores, dam and bridges are accessible to public.

Future care and maintenance of the work:

National Capital Authority is responsible for on-going operation and maintenance.

Name of Sponsor:

Heritage Panel, Canberra Division, IEAust.

For a NEL, is an information plaque required?

Yes.

....

Nomination prepared for Engineering Heritage Panel
by B.W. Kenny BEc. MIEAust

....

Chairperson of Division Heritage Panel

ADDITIONAL SUPPORTING INFORMATION

Name of work:
Lake Burley Griffin Scheme

Year of construction or manufacture:
1957 – 1963

Period of operation:
37 years

Physical condition:
Excellent – fully operational

Engineering Heritage Significance:-

Technological scientific value:
Highly effective application of engineering science which demonstrates measures appropriate to flood control, environment protection land uses for artificial lakes.

Historical value:
High – a key feature of national capital plan adopted soon after federation.

Social value:
Focus for public events, recreation, tourism in the city of 300,000.

Landscape or townscape value:
Outstanding setting for national institutions.

Rarity:
No other similar scheme for this purpose in Australia.

Representativeness:
Most effective combination of engineering with other disciplines to give visual symbolism and ecological safeguards.

Contribution to the nation or region:
Integral part of nation's capital, and important ecological resource in upper Murrumbidgee catchment.

Contribution to engineering:
Major public demonstration of engineering.

Persons associated with the work:
See section 4 attached.

Integrity:
100% - operating as constructed.

Authenticity:
Unchanged

Comparable works
(a) in Australia:
None

(b) overseas:
Comparable artificial water features in national cities are not known.

Statement of significance, its location in the supporting documentation:
See Section 3.

Citation (70 words is optimum):
Attached in Section 11.

Attachments to submission (if any):
As scheduled in Table of Contents.

Proposed location of plaque (if not at site):
Owner's policy requirements are to be discussed.

NOMINATION OF LAKE BURLEY GRIFFIN SCHEME IN CANBERRA, AUSTRALIAN CAPITAL TERRITORY AS A NATIONAL ENGINEERING LANDMARK

TABLE OF CONTENTS

1. INTRODUCTION

The Lake Burley Griffin Scheme is an engineering work which implements the aesthetic concepts central to Australia's national capital's functions and symbolism. The Lake unites the city and creates a setting for one of the most beautiful national capitals in the world.

This nomination examines the three main components, namely:

- Scrivener Dam - the impounding and flood control structure
- Lake Burley Griffin and foreshores - visual focus of landscaped setting for national buildings and public social and recreational uses
- Bridges at Commonwealth Avenue and Kings Avenue - major crossing points in the central city precincts and transport linkages uniting the city

It is a major engineering contribution to the city's plan and a visually dramatic feature of the national capital which can "excite the imagination and raise the vision of its people". (Priddle, quote from Third Annual Report of National Capital Development Commission)

2. LOCATION AND PHYSICAL SETTING

Lake Burley Griffin and its catchment are located within the upper Murrumbidgee River catchment on the Southern Tablelands of New South Wales (Figure 1).

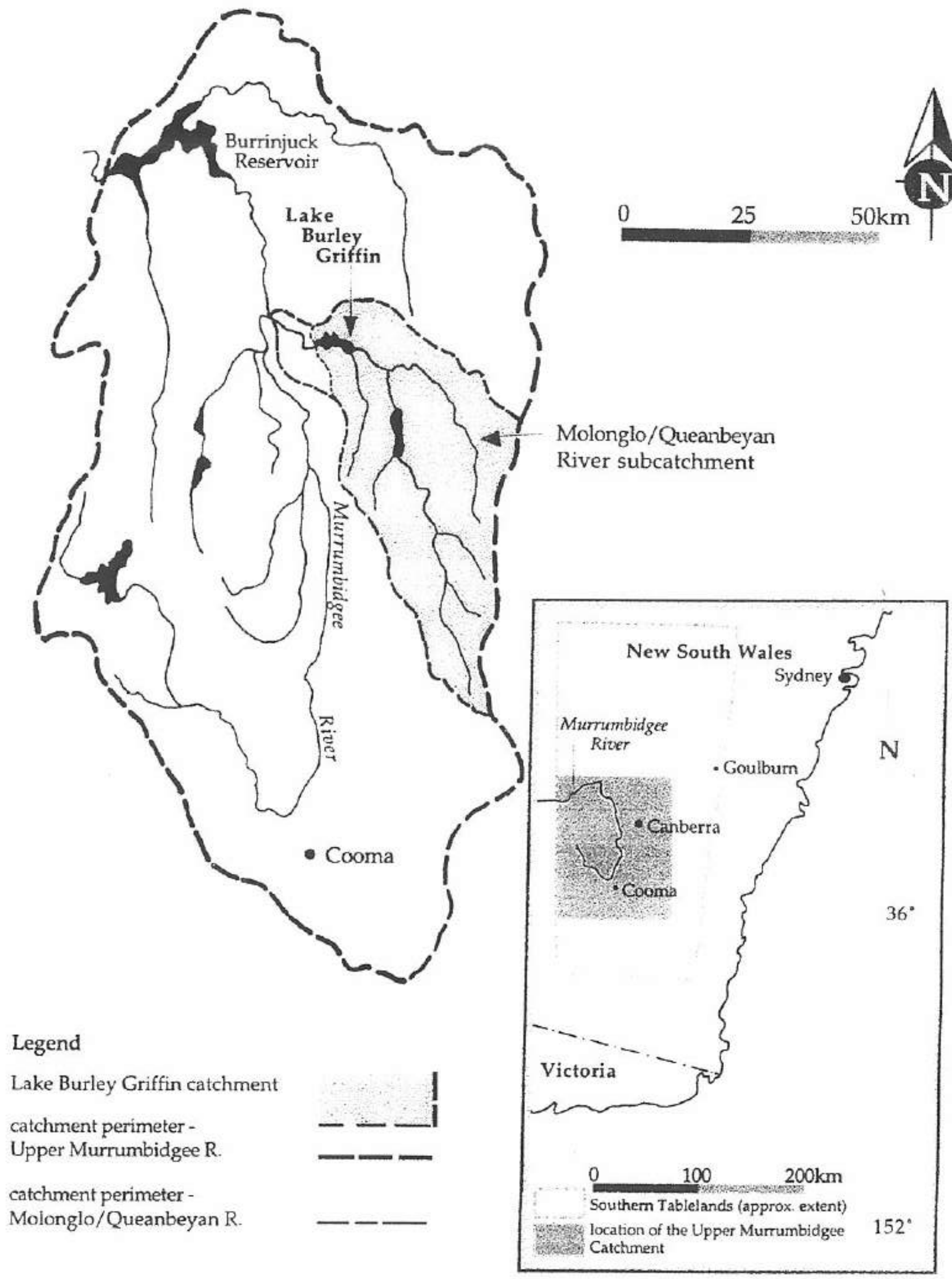

Figure 1: Location of Lake Burley Griffin catchment within upper Murrumbidgee River catchment.

3. STATEMENT OF SIGNIFICANCE

Walter Burley Griffin's plan for Australia's federal 'seat of government' was based on a central water feature. The scheme (dam, lake and bridges) was implemented by the newly established National Capital Development Commission (NCDC) in 1957-63, as one of its first major projects.

The Lake has important symbolic values, since it defines the origin point for the city and nation. The Central Basin is the centre piece of the National Area, in the immediate foreground of the Parliamentary Zone, and unites the wider city-scape and public spaces on its foreshores in which are set major national buildings. (National Capital Planning Authority 1995).

The project involved high quality engineering solutions to aesthetic and land use needs, and visual requirements for bridges to complement future national buildings. (supporting documents in Section 7)

The water surface and foreshores provide a city of 300,000 with avenues for recreation, social and public events and community gathering and present easy access to national buildings set in adjacent landscape. Water level is maintained by specially designed flood control gates in Scrivener Dam, in a rare application in Australia. (Smith and Coltheart, and C.J. Price)

In ecological terms, the lake has created valuable wetland habitats, offers some protection against excessive siltation downstream, and the Dam fills an important flood control function (NSW Department of Land and Water Conservation – 2000).

The dam component of the scheme has been nominated for the Register of the National Estate. (Smith & Coltheart) and is rated 5th of 25 Australian Heritage Dams in the 1999 Institution of Engineers, Australia report to the Australian Heritage Commission.

The completed scheme symbolises the development of the nation and is a significantly outstanding engineering contribution to the Federal Capital prescribed in the Constitution.

4. ASSESSMENT OF HERITAGE SIGNIFICANCE

- National – the attached extract from the National Capital Authority's "Lake Burley Griffin Plan of Management Issues Paper" summarizes its significance under the section "The National Significance of Lake Burley Griffin"

- the scheme is a unique example of engineering and associated technology being used to create a major project whose principal objectives are aesthetic. Up-to-date technology and engineering science including hydraulic modelling, environmental analyses and contemporary practices on concrete design and construction were used extensively in the implementation phase. The completed project brings into focus the natural beauty of distant mountains and adjacent hillsides and visually unites them with a created setting for public institutions, complemented by low profile bridges.

- historic – the concept of a water feature as a major element in the future city was identified in earliest consideration of the site. Its subsequent progress to completion moved with the social, economic and political development of the nation, and associated advances in engineering practice.

- social – the scheme's completion gave a focus for social, recreation and tourist activities in Canberra – the water and surface and public areas of the foreshores have encouraged water sports, cycling and walking, and provide community gathering places for public occasions, and the foreshores and easy access to public buildings present clear identification of Canberra as a national city.

- regional ecological benefits and protection downstream against flood flows have followed completion of the scheme. NSW Department of Land and Water Conservation (2000) notes these important benefits and valuable wetland habitats.

- significant personalities are associated with the scheme's evolution and construction. Charles Scrivener, who surveyed the proposed site in detail before it was finally selected, and Walter Burley Griffin (the city's original planner) are commemorated by name. Many notable engineers and other professionals and public figures have been involved:
    - Sir John Butters (Commissioner of Federal Capital Commission)
    - Sir John Overall (Commissioner of National Capital Development Commission)
    - William C Andrews (Commissioner NCDC)
    - Marion Mahony Griffin (notable in early planning) produced drawings and architectural presentations to support her husband's concepts and early plans of the city.
    - Minister King O'Malley initiated action soon after Federation, Prime Prime Minister Sir Robert Menzies ensured adequate funding and political commitment to completion.

- in heritage ranking of Australian dams, the scheme's impounding Scrivener Dam is ranked 5 (of 25 recommended for inclusion on Register of National Estate) – appendix H of Report (26.1.99) to Australian Heritage Commission.

THE NATIONAL SIGNIFICANCE OF LAKE BURLEY GRIFFIN

A Feature of the National Capital

Lake Burley Griffin is one of the most important landscape features of Canberra in its role of the National Capital, being envisaged as a key element in the plans of Walter Burley Griffin and eventually being implemented as one of the first major projects of the National Capital Development Commission. Not only is it the centrepiece of the Central National Area but it also forms the immediate foreground of the Parliamentary Zone.

The present lake and foreshores have not been constructed entirely in accordance with Griffin's plan (see Figure 3). Griffin envisaged a formal central basin on the main land axis of the city with two complementary basins at each end forming his 'water axis'. The basins as constructed do not display the artificial symmetry of Griffin's design although a level of formality, particularly in Central Basin is clearly evident.

Downstream of West Basin, Griffin proposed West Lake which in the ultimate plan was enlarged considerably to extend to Scrivener Dam. Upstream of East Basin, he proposed an irregular 'East Lake' formed by the Causeway (an extension of the existing street by that name) which would carry railway and road connections between the northern and southern sides of the lake. East Lake would have flooded the Dairy Flat - Pialligo area.

The plan for Lake Burley Griffin was in fact amended several times from Griffin's plan before it was constructed, its current form being strongly influenced by the 1959 ideas of eminent British planner, Sir William Holford. It is nevertheless clearly identifiable with much of Griffin's concept. Griffin, however, never intended the lake foreshore to be a uniform ring of parkland without buildings or crowd activities. He proposed monumental groups of buildings along the southern shore of the Parliamentary sector and cultural buildings surrounding his proposed amphitheatre on the West Lake esplanade along the waterfront in the Yarralumla area. He also proposed boulevards around the central basins which would conform with the architectural character of the area and concentrate activity and movement along the shore of the lake.

Many such details of the Griffin plan have not been implemented, although the National Capital Plan maintains Griffin's principle of using Lake Burley Griffin as the major landscape feature which unifies the National Capital's central precincts and the surrounding inner hills. Policies in the *National Capital Plan* for Lake Burley Griffin and its foreshores include the following (NCP, p. 20):

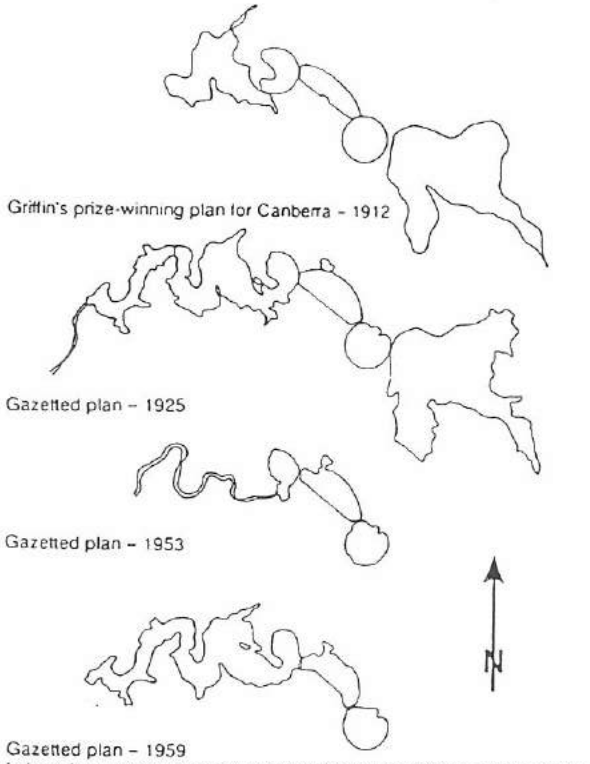

Figure 3. Evolution of the plan for Lake Burley Griffin

- *Lake Burley Griffin and Foreshores should remain predominantly as open space parklands while providing for existing and additional National Capital and community uses in a manner consistent with the area's national symbolism and role as the city's key visual and landscape element.*
- *Lake Burley Griffin and Foreshores are intended to provide a range of recreational, educational and symbolic experiences of the National Capital in both formal and informal parkland settings with particular landscape characters or themes. These should be maintained and further developed to create a diversity of landscape and use zones which are integrated into the landscape form of the city and reflect the urban design principles for the National Capital.*

5. ENGINEERING HERITAGE BACKGROUND

In the conventions and public discussion during the 1890's between the various advocates of federation for Australia it was recognized that a seat of central government, separate from each colonial capital, was needed. The Constitution Act which came into effect on January 1901, provided for this in section 125 – a seat of government was to be built on land excised from New South Wales, at least 100 miles from Sydney.

In his Presidential address (copy attached in Section 7) to the 44th Annual General Meeting of The Institution of Engineers, Australia (April 1964), R.A. Priddle noted the 1909 report by surveyor Charles Scrivener of N.S.W. Lands Department recommended the Molonglo basin near Yarralumla as a site for the federal capital and made the first known reference to ornamental water for the city. This was mentioned in the rules for an international competition conducted in 1911 for the design of a National Capital.

C.J. Price (then First Assistant Commissioner, Engineering for National Capital Development Commission) in his chapter on "Lakes and Dams", in "Canberra's Engineering Heritage" (copy attached in Section 7), recorded that "An engineer, J.A. Smith, was one of the majority of judges who awarded first prize to Walter Burley Griffin in 1912 for his entry in the Federal Capital Design Competition. Griffin had placed the central basin of his lake scheme across the land axis of his design, with two formal basins at each end forming his "water axis" _ _ _ _ he grasped, as his rivals and critics had not, the significance of the Molonglo flood plain".

The other two judges (an architect and a surveyor) were divided about Griffin's entry, and Minister King O'Malley accepted the majority view. (Sir John Overall – Memoirs)

Engineering research and analyses over the next 50 years considered how to implement Griffin's concept of central unifying bodies of water in the Molonglo flood plain..

Between and 1920's and 1950's, a number of reports to parliament were prepared on the form of future development of Canberra – by then the established seat of Government with provisional Parliament House opened in 1927 – and hydrological data was being collected about the Molonglo River flows. Engineers had major inputs to those reports (in 1929, 1936 and 1955) notably W.E. Potts and Percy Owen (Chief Engineer of Federal Capital Commission under Sir John Butters and previously Director General of Works during the Design Competition). Also they were the first Chairmen of Canberra Division.

In 1957 the government established the National Capital Development Commission, which was assisted by its National Capital Planning Committee (non-Commission members, including 2 engineer representatives – refer Priddle above). N.C.D.C. quickly recognized the importance of the lake, and was able to draw on the earlier studies and on the technical resources and hydrological data available through Commonwealth departments and authorities regarding the behaviour of the lake in terms of floods and droughts and of scour and siltation. Other studies provided satisfactory answers on water quality, effects of climate and health, hazards of unsightly margins, of mosquitoes and midges and the possible disbenefits from changes in land uses". (refer C.J. Price, above)

Price describes the essential hydrological issues of the lake and dam design and criteria used in design and operation of the "fish belly" control gates in this rare application in Australia.

Sir John Overall (NCDC Commissioner) records (P.56 of Memoirs), with regard to the dam "Faults in the foundation bed-rock of the river-bed at the dam site could have substantially delayed or increased the cost of the structure, but for post-tensioning techniques only recently developed. It also would not have been possible for the engineers to have so accurately maintained control of the water at the 556 metre level (1825 feet) determined by the model test, were it not for new "fish belly" crest gates built in Germany. These gates also freed the dam from an awkward superstructure treatment, and allowed a road to run over it".

In 'Canberra's Engineering Heritage' NCDC Associate Commissioner, W.C. Andrews, comments about Commonwealth Avenue bridge "Noteworthy in itself, especially in a heritage sense, was the technology involved in the pre-designed method of construction of the concrete box super-structure." The Prime Minister, Sir Robert Menzies, in October 1964 described the bridge as "the finest building in the National Capital".

The original concept of Burley Griffin's series of formal basins with informal bodies of water up-stream and down-stream, became one large lake (with surface level as suggested by Scrivener 50 years before), impounded and controlled by a single dam structure, and crossed by two major bridges.

The scheme's final form and its technical viability, depended on excellence in engineering analysis and judgement and application of best practices available.

6. SUMMARY OF ENGINEERING FEATURES

The Lake is formed in the flood plain of the Molonglo River, impounded by Scrivener Dam, and unifies the central precincts and national areas of Canberra. The extent of the lake and its relationship to principal roads and public buildings and spaces are shown on Figure 4 "Lake Burley Griffin" , available by courtesy of National Capital Authority.

Lake management is a Commonwealth responsibility administered by the National Capital Authority, with day-to-day management undertaken by arrangements with various agencies within A.C.T. Territory Government. Catchment protection in NSW was undertaken until 1998 under a Commonwealth -State Agreement.

The Lake surface is maintained at 556 metres TWL (1825 feet) above sea level subject to temporary variations due to flood management requirements and other catchment inflow effects. Lake volume is 33,000 ML and surface area at T.W.L. is 634 hectares. Foreshore length totals 33 kilometres, most of this is available for public access and recreation.

Scrivener Dam is a concrete gravity dam, 33 metres high and 235 metres long. Its spillway consists of 5 bays of free overflow crest, controlled by hydraulically operated "fish belly" flap gates, with capacity of 8,500 cub metres per second. It is one of the largest of this type of control, rare in Australia. Figure 2 "General Arrangement of Dam" is by courtesy of National Capital Authority.

Details of main aspects of design and operation of the Lake and dam are presented in attached 1964 Paper and Discussion by A.J. Condon, B.V. Kearsley and A. Fokkema, titled "River Model, Spillway Model and Dam Design for the Canberra Lake Scheme".

The dam structure incorporates a roadway which provides one of three crossings of the lake. The others, at Commonwealth and Kings Avenues, are major bridges. Each is a twin structure, with separate parallel carriageways. Details of design of these major bridges are given in attached 1964 Paper and Discussion by E.M. Burkett and G.N. Fernie.

The Commonwealth Avenue bridge structures are of 5 spans totalling 310 metres, of fully continuous prestressed design. To provide an exit and entry clover leaf layout, separate structures are built on the main bridge's south approaches, totalling 49 metres each, in 4 approximately equal spans. The bridge was opened in November 1963.

Kings Avenue parallel structures use longitudinal precast prestressed concrete T beams forming separate carriageways, each of 7 spans totalling 270 metres. Underpass roads for exit and entry are provided at south end only within the bridge spans. The bridge was opened in March 1962.

7. SUPPORTING DOCUMENTS

- PRESIDENTIAL ADDRESS "HISTORY AND DEVELOPMENT OF AUSTRALIA'S NATIONAL CAPITAL"
  by R.A. Priddle – April 1964 Annual General Meeting of The Institution by Engineers, Australia.

- LAKES AND DAMS"
  by Clive J Price in Canberra's Engineering Heritage 2nd Ed. 1990.

- "RIVER MODEL, SPILLWAY MODEL AND DAM DESIGN FOR THE CANBERRA LAKE SCHEME"
  by A.J. Condon, B.V. Kearsley and A. Fokkema in Journal of The Institution of Engineers, Australia – September 1964.

- "BRIDGES IN THE CANBERRA CENTRAL LAKE AREA - DESIGN"
  by E.M. Birkett and G.N. Fernie in Journal of The Institution of Engineers, Australia – July – August 1964.

- "SCRIVENER DAM – NOMINATION FOR LISTING ON THE REGISTER OF NATIONAL ESTATE:
  Prepared by Mike Smith, reviewed by Lenore Coltheart – January 1998 – relevant extracts

Presidential Address*

History and Development of Australia's National Capital

BY R. A. PRIDDLE, B.E., M.I.E.AUST.

Introductory

After some twenty years of argument, wrangling and final agreement between the various colonies, the Commonwealth of Australia was born, in an atmosphere of general rejoicing, on 1st January, 1901.

Although general agreement had been reached and the new nat‐ had a written constitution, a site for the national capital ha‐ ‐ot been chosen. Every city and town of any consequence had put forward strong claims for the honour, but section 125 of the Commonwealth of Australia Constitution Act, product of a last-minute compromise between the Premiers of New South Wales and Victoria, provided :

> " The seat of Government of the Commonwealth shall be determined by the Parliament, and shall be within territory which shall have been granted to or acquired by the Commonwealth, and shall be vested in and belong to the Commonwealth, and shall be in the State of New South Wales, and be distant not less than one hundred miles from Sydney.
>
> Such territory shall contain an area of not less than one hundred square miles, and such portion thereof as shall consist of Crown lands shall be granted to the Commonwealth without any payment therefore.
>
> The Parliament shall sit at Melbourne until it meet at the seat of Government ".

Another ten years of investigation, argument and more investigation were to elapse before a choice was made.

In 1908 Mr. Charles R. Scrivener, a Surveyor in the New South Wales Lands Department, was instructed to make a detailed reconna‐ nce of the Yass-Queanbeyan area and in February, 1909 he pr‐‐nted a report which recommended the Molonglo basin near Yarralumla. In his report the scenic and climatic advantages were stressed, and he pointed out that the river valley lent itself to the formation of artificial lakes as a central feature of the city-to-be. This major contribution, the first known reference to ornamental lakes, was recognised earlier this year when the dam which forms Lake Burley Griffin was named Scrivener Dam.

Scrivener's report was finally accepted, the Commonwealth took possession of the Australian Capital Territory—of nearly one thousand square miles—on 1st January, 1911, and in the same year an international competition was called for the design of a National Capital. The possibility of forming a lake was stated thus in the competition rules.—

> " 6. *Ornamental Water*.—Two sites for weirs across the Molonglo River have been examined—one at the rocky bar, almost in line between trigonometrical stations 'Sullivan' and 'Shale', and the other beyond the western boundary of map ".

It is interesting to note that Scrivener favoured the second site and suggested a top water level of R.L. 1825. His choice of site and lake level were confirmed by the engineering investigations which led to their adoption in 1958.

The competition, like so many other things to do with the National Capital, was strongly criticised in some quarters, and the Royal Institute of British Architects advised its members and affiliated bodies throughout the world not to participate because the final choice of design lay with the Minister—a layman in architectural matters. In spite of this one hundred and thirty-seven designs were received.

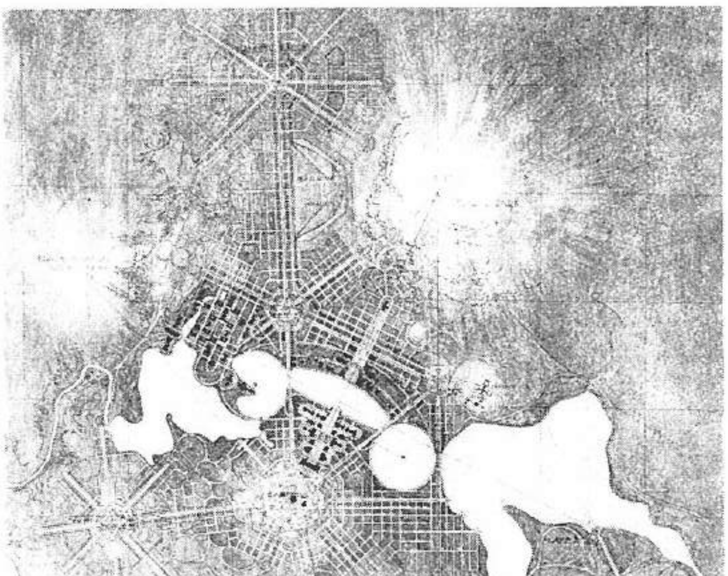

Fig. 1.—Reproduction of the Original Burley Griffin Plan, 1911.

The winning design, that of Walter Burley Griffin, a landscape architect from Chicago, turned the " ornamental water " into a central group of three formal basins together with an informal extension westward (downstream) to the vicinity of Yarralumla, and another informal lake to the east, which had a slightly higher surface level. Lake Burley Griffin, as built, has three rather less formal central basins and a larger informal lake to the west. The lake at the eastern end was found to be impracticable.

*To the Forty-Fourth Annual General Meeting of The Institution of Engineers, Australia, at Canberra on 16th April, 1964.

Fig. 2.—Statutory Plan of Canberra, 1955.

In his winning design (see Fig. 1) Griffin took as his main feature the Land Axis from Mt. Ainslie to Capital Hill, crossing the formal central basin at right angles and bisecting an equilateral triangle which linked Capital Hill, City Hill and a third high area now known as Russell. Apart from the siting of a few major buildings Griffin did not attempt to define the land use within the City. It was a geometrical picture, formed by lakes, streets, railway lines, avenues and vistas set in the broad Molonglo Valley and framed by the surrounding hills. It was certainly not a "town plan" in the modern sense, but it made full use of the natural beauty of the site.

Early Development.

Burley Griffin:

Griffin's plan and three other competition entries were submitted to a technical Board for review, but the Board prepared a fifth plan, incorporating various features from each design. When Lady Denman named the new city "Canberra" at a special ceremony at Capital Hill on 12th March, 1913, the compromise plan was the basis for laying out the city. However, a new Government invited Griffin to Australia for discussions with the Board. Agreement proved impossible, the Board was disbanded and Griffin was ointed Federal Capital Director of Design and Construction, a post within the Department of Home Affairs which he held until 1920.

In spite of criticism, shortage of funds and the effect of World War I Griffin was able to establish on the ground the main framework of his design for Canberra, with minor modifications found necessary as detailed investigation proceeded. The main features of his 1918 plan are very much in evidence today.

Federal Capital Advisory Committee:

A Federal Capital Advisory Committee of engineers and architects, with Sir John Sulman, F.R.I.B.A., M.T.P.I. as chairman, was appointed in 1921 "with a view to enabling the Federal Parliament to meet and the Central Administration of the Commonwealth Government to be carried out as early as practicable at Canberra (and on the basis of the acceptance of the plan of layout of the Federal Capital City by Mr. W. B. Griffin)".

The Committee recommended drastic curtailment of Griffin's plan, but this was rejected by the Government, and Griffin's design has never since been seriously challenged. The Committee had not the powers nor the financial resources to prosecute construction of the city vigorously, and in 1925 it was succeeded by the Federal Capital Commission.

Federal Capital Commission:

From 1925 to 1930 the Commission, under Sir John Butters, C.M.G., M.B.E., M.I.E.Aust., was able to achieve a great deal in the development of the growing city. During this period the provisional Parliament House was completed, the Administrative Block commenced and engineering services completed for a city of 25,000 people. This population was not reached until 1952. Sir John was President of The Institution in 1927.

The average annual expenditure of £2,000,000 raised mostly by loans was considerably greater than in earlier years, the progress being correspondingly accelerated. However, the depression led to reduced activity and a new Government in 1930 abolished the Commission and handed the administration back to the Department of The Interior.

National Capital Planning & Development Committee:

Following severe public criticism in 1938 the Government established the National Capital Planning and Development Committee to advise the Minister of The Interior on matters referred to it, with power to make independent recommendations on its own initiative. This Committee of architects and engineers plus three departmental officers, with Mr. B. J. Waterhouse, O.B.E., F.R.I.B.A., F.R.A.I.A. as Chairman, met monthly for almost twenty years. It made many worthwhile contributions to the development and beautification of the City, but its powers were limited and its recommendations were sometimes overruled. This difficult was emphasised when the Department of Works separated from th Department of the Interior and administration and constructio became the responsibility of two different Departments.

Fig. 2 shows the Statutory Plan of Canberra including all amend ments to 1955. Note the omission of the East and West Lake with a "ribbon of water" at the western end, to be formed by low level weir near Yarralumla.

Senate Select Committee:

In 1954 the Government appointed a Select Committee of tl Senate, under the Chairmanship of Senator J. A. McCallum, "enquire into and report upon the development of Canberra relation to the original plan and subsequent modifications, ar matters incidental thereto". The report of this Committee, pul lished in 1955, gave a comprehensive picture of the developme of the National Capital, and recommended the establishment a single Canberra Authority under a Minister for the Australi Capital Territory.

The Government later invited the eminent Town Plann Sir William Holford, A.R.A., F.R.I.B.A., M.T.P.I., Professor Town Planning, University College, London, to visit Canbe and to advise on the planning of the city, and his report was cor pleted in December, 1957.

In his "observations" Holford drew attention to the ma difficulties inherent in Griffin's plan, viz.:

1. The great distances between the three focal points of the trian (at that stage reduced to two by the absence of any developm at the third point of the triangle).
2. The disruptive effect of the Molonglo flood plain forming a barr between the northern and southern parts of the city.
3. The street pattern was completely unsuitable for modern traf
4. The land axis needed better definition in its middle length.
5. Complete symmetry was no longer feasible.

Holford favoured the early establishment of the Lakes, i cluding the contentious West Lake, with suitable features on t lake side to make the lakes a unifying central feature of the c rather than a barrier between two independent towns. He al suggested important modifications to the street pattern, providi for throughways and parking areas. To improve internal co munication he suggested a curved "parkway" near the shore the lake, joining the eastern ends of Commonwealth Avenue a Kings Avenue Bridges. He also put forward convincing argume in favour of the permanent Parliament House being situated Griffin's Water Gate, centrally on the Southern shore of the Cent Lake. This point should become a visual centre of the new unified city, as well as having considerable practical advantag as compared with the other possible site on Capital Hill. Fig. illustrates the main features of Holford's "observations". W Lake is tentatively restored, but the eastern basin is curtail Traffic flow is improved.

National Capital Development Commission:

In September, 1957, the National Capital Development Co mission Act was passed, and the National Capital Developme Commission was constituted, the Commissioner being Mr. J. Overall, C.B.E., M.C., F.R.I.B.A., F.R.A.I.A.

The Commission is responsible for planning, design a construction of the City of Canberra, and to that end it has wi powers within the Australian Capital Territory. Property contr administration and survey remain functions of the Department The Interior, with which Department the Commission has a ve close liaison. The Department of Works continues to act as Co structing Authority for much of the Commission's constructio but a considerable amount is done by the Commission throu contracts administered by private consultants. The Commissio expenditure of £11,000,000 in 1958-59 has gradually been increas to keep pace with the accelerated population growth, which has be much faster than was originally forecast. In the last financial y the Commission spent £14,000,000 out of a total expenditu government and private, of £28,000,000.

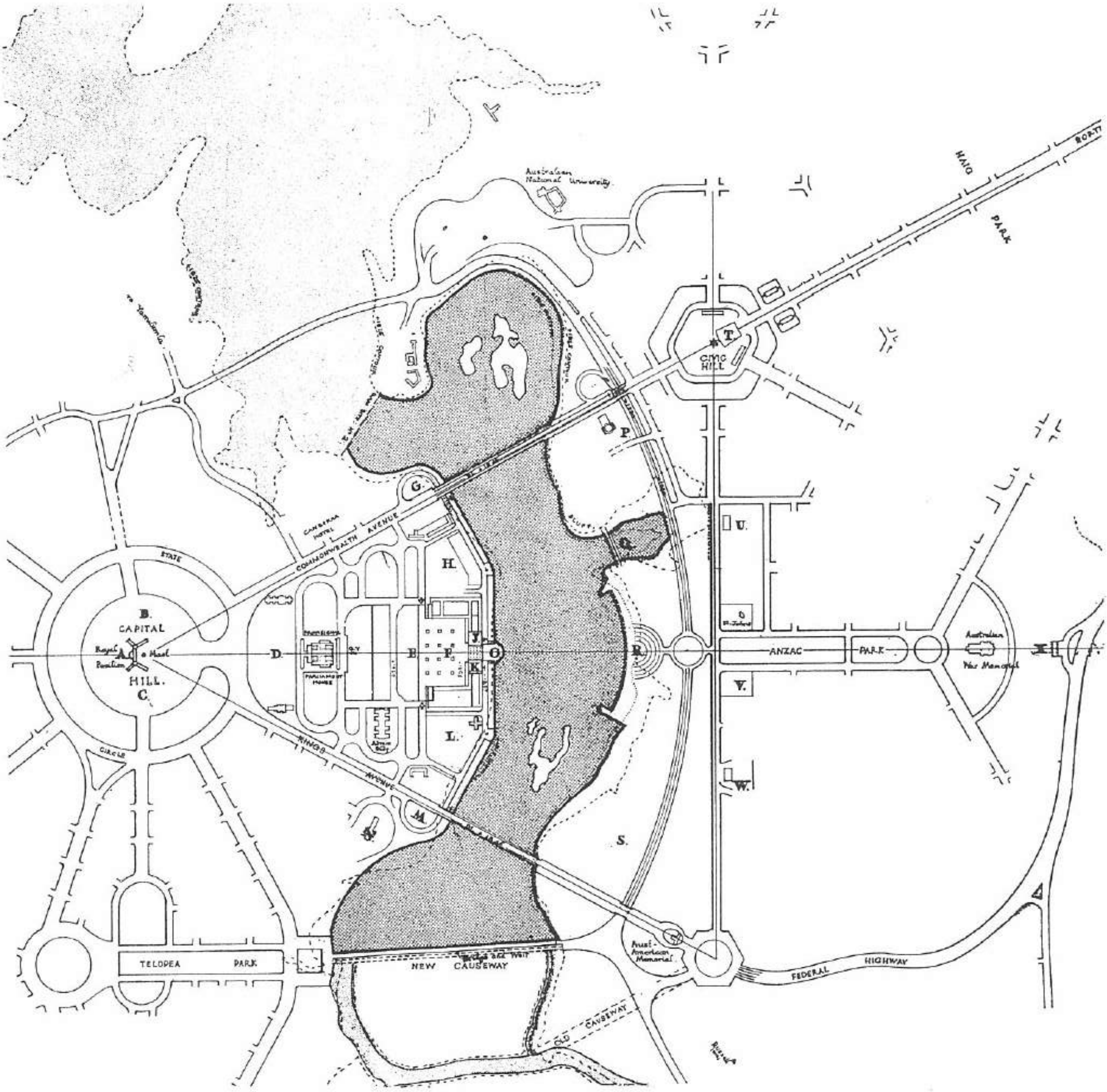

Fig. 3.—Sketch accompanying Sir William Holford's " Observations ", 1957.

In matters of planning and design the Commission is assisted by the National Capital Planning Committee which meets monthly. The Committee consists of two representatives of the Royal Australian Institute of Architects, two from The Institution of Engineers, Australia, two from the Australian Town Planning Institute and " two other persons with special knowledge and experience in artistic or cultural matters ". I had the honour to be one of The Institution's representatives from the commencement in 1958 until 1962.

In addition to examining and assessing a wealth of data—much of it no longer relevant—on the planning of Canberra, the Commission was faced immediately with the problem of providing, within 18 months, office and residential accommodation of the defence departments which were being moved from Melbourne. With families, tradespeople and ancillary services this move was responsible for an increase in Canberra's population of about 5,000 people in this short period. Some major policy decisions were taken very early, viz. :

1. All new office buildings and dwellings would be of permanent construction, rather than makeshift or " temporary " buildings.
2. The Commission would encourage private enterprise development to the fullest extent, in order to reduce the demand for Government expenditure.
3. Planning would be directed towards achieving a properly balanced development.
4. The Commission would aim at a stable rate of expenditure from year to year, although subject to the usual budgetary control.

The last feature, which was sadly lacking in Canberra between the wars, has contributed greatly to stability in the construction industry and has encouraged major contractors to seek work in the Capital Territory.

Another important early step was the establishment of offices for the service Departments and the Department of External Affairs at Russell, near the Australian-American War Memorial. This brought to life the " third point " of Griffin's triangle and helped with traffic problems, at the same time changing the emphasis on some aspects of Holford's " observations " and other earlier studies.

Fig. 4.—Plan of Canberra, 1964.

The Commission was able to endorse most of Holford's suggestions, and when its first report was adopted in principle by Cabinet in July, 1958, the way was clear for more detailed planning to proceed.

Progress 1958–1964.

Since taking office in 1958, when Canberra's population was 35,000, the National Capital Development Commission has expanded the statutory plan and modified it where necessary to accommodate a population approaching 250,000. This has involved the development of many new suburbs and a completely new district (Woden or Yarralumla Creek Valley) as well as forward planning for other districts such as Belconnen. The present population of Canberra is about 70,000.

The example set by the Commission, together with active encouragement of private investment, has generated confidence in and enthusiasm for the future of Canberra to such an extent that private dwellings now represent more than half the total dwellings under construction, while private investment in offices, stores and other commercial buildings is running at a record level. Total expenditure on all private buildings is approaching £14,000,000 per annum.

rom the Commission's expenditure it has provided many public buildings, including the office buildings at Russell, Civic Offices, Auditorium and Supreme Court at City, Government Printing Office, Tariff Board and others. The National Library and a major building for the Bureau of Census and Statistics are under construction in the "Government Triangle" while the National Mint is well advanced in the new district of Woden. Buildings housing electronic data processing equipment for various departments are under construction, while schools have been erected in most of the suburbs. The Commission has also administered much of the construction programme of the Australian National University, as well as playing a major part in the siting and design review for all major buildings, public or private, in Canberra.

Engineering services have accounted for a large proportion of the annual expenditure. Major units completed include:

Bendora Dam (water supply) on the Upper Cotter River.
Scrivener Dam near Yarralumla
Lake Burley Griffin and surrounding treatment
Kings Avenue Bridge
Commonwealth Avenue Bridge
Parkes Way, together with many miles of streets, motorways, improved intersections and sign-posting
Extensions of all engineering services, including roads, water supply, sewerage, drainage and power for extensive new suburbs.

ig. 4 shows the plan of Canberra as it is today.

The Future.

In the next decade the population will reach 150,000. Present residential areas will have been completely developed to house about 90,000 and most of the remaining 60,000 will be living in the Woden district. Belconnen will be in about the same stage of development as the Woden Valley is today.

A gravity water main will have been completed between Bendora (upper Cotter) Dam and the city, together with a new water supply dam farther up the Cotter River. All water will be fully treated before it is reticulated to consumers.

Existing sewage treatment works at West Creek will have been extended to their practical limit, and supplemented by a modern new treatment works near Fyshwick—the effluent being piped to an outfall well downstream of Scrivener Dam.

Traffic flow between the Woden Valley and the northern areas will demand consideration of a further bridge across West Lake, about half way between Commonwealth Avenue and Government House. Some form of rapid transit will probably be in operation between major city centres.

The National Library and other major public buildings will be complete. It is hoped that planning for the permanent Parliament House on the lake shore will be well advanced, and that the National Galleries of Art, Science and History on Capital Hill will be a major tourist attraction.

The next milestone—a population of 250,000—should be reached between 1985 and 1990. By this time all major buildings at present envisaged in the Parliamentary triangle will be complete, Woden and Belconnen will be fully developed, together with further major areas such as the Majura Valley. The latter development will require a new airport at Mulligan's Flat, north of the city. Car parking—if motor cars are still a major factor in city life—will be in multi-storey car parks, and high-rise buildings will occupy many of the present open spaces. It is inevitable that as the city grows many aspects of administration will be transferred to a local government body having wider powers and greater responsibilities than the present Advisory Council.

Current planning has not gone beyond the stage of 250,000 population, but Professor W. D. Borrie, M.A., Professor of Demography, Australian National University, estimates a population of 500,000 by the end of the century. We can expect that the planning for this expansion will be undertaken in good time. Detailed planning should not extend too far ahead because of the possibility of unforeseen changes in the pattern of living, induced by technological developments or other causes.

I will conclude by quoting from the third Annual Report of the National Capital Development Commission.—

"No city can become a National Capital of any significance unless it can excite the imagination and raise the vision of its people. Other capitals do this by providing a fine setting in the central areas for the main institutions of Government—the Parliament, the Courts, the Mint, National Libraries, National Museums. Within these and other buildings, and in their garden settings, they enshrine their National history, as Canberra has done, for example, in the National War Memorial. Great events and great men are commemorated by features which enhance the city. This is well illustrated in the United States where Washington has become in every sense a great National Capital because it is a true home of Congress. It enshrines the nation's history and it offers great interest to millions of Americans who visit it each year to see the national shrines and visit the institutions of Government.

"A National Capital depends on national sentiment. While it is not possible to create sentiment with money, it is equally impossible for sentiment to develop around a vacuum. To create these things which arouse curiosity and interest, both imagination and expenditure are required".

Tonight we are meeting in a living metropolis where less than fifty years ago there was nothing but broad acres—in a national capital which has doubled its population in the last five years, which will again double its population in the next decade and may continue to do so for several succeeding decades.

To us in the engineering profession and to our colleagues in the architectural profession this alone is an exciting challenge.

The architectural, engineering, artistic and cultural developments that must and will keep pace with population growth cannot help but excite the entire Australian nation.

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Chapter Four

LAKES AND DAMS

Clive J. Price MBE, BE,
FIE Aust., Hon. FRAPI

The author has been actively engaged in the development and management of water resources for military, municipal, and recreational purposes and for hydro-electric development over the past forty years. Between 1958 and 1972 he was First Assistant Commissioner, Engineering of the National Capital Development Commission during which many of the major works, including Lake Burley Griffin, were undertaken. For the subsequent [illegible] years he was a Director of consulting engineers, M[illegible] and Partners, also working on major development pr[illegible] Canberra.

ONE of the many definitions of engineering suggests that it is the application of available resources for the benefit of man. In the construction of Canberra's lakes and dams over the past decades, the Territory's water resources have been developed to provide an increasing range of benefits, initially for the supply of adequate quantities of drinking water of acceptable standards (Cotter, Corin, Bendora and Googong Reservoirs), and with Lake Burley Griffin providing an ornamental setting of great beauty for the Capital and a recreational facility of inestimable value. Lake Ginninderra, adjacent to the Belconnen Town Centre, also provides a pleasant amenity and recreational facility for residents on a smaller scale as well as providing a degree of environmental protection for the Murrumbidgee River.

An assured and adequate water supply and beauty of the site were factors in the selection of the Canberra area as the site for the National Capital. Scrivener who inspected the district in 1909 was impressed by the opportunity it afforded for 'storing water for ornamental purposes at reasonable cost'. In making his choice, Scrivener unknowingly selected the spot where, in pleistocene time, a freshwater lake had existed, created when scree from Black Mountain blocked the channel of the Molonglo River, damming the water back to a height of about 556 m above sea level. In his 1909 contour survey, Scrivener showed four alternative weir sites for the construction of an ornamental lake with a water level of 556 m. These were the first schemes for a lake at Canberra and the concepts were similar in size and shape to the lake as it exists today.

In determining that the future Capital would have adequate water supplies, Scrivener proposed that the Capital Territory should include the catchments of the Molonglo and Queanbeyan Rivers to prevent pollution of the rivers before they flowed through the city site. Legislation to create the Territory was passed by the State and Commonwealth Governments, but the Molonglo and Queanbeyan River catchments were excluded.

However, the Seat of Government Act which led to the establishment of the Capital Territory on 1 January 1911 gave the Commonwealth paramount rights over the Molonglo and Queanbeyan Rivers and their tributaries, and made the State of NSW responsible for protection of the river waters from pollution. The new Territory also included the catchment of the Cotter River and it was to be on this river that three of Canberra's four water supply storages, Cotter, Corin and Bendora were to be constructed.

The wisdom of the 'Founding Fathers' has enabled a substantial heritage to build up through the lakes and dams which have emerged as the rivers and streams have been developed for water supply, for active and passive recreation, for water quality control, for town centre cooling and as an integral part of the complex planning of a National Capital. The significance of Lake Burley Griffin

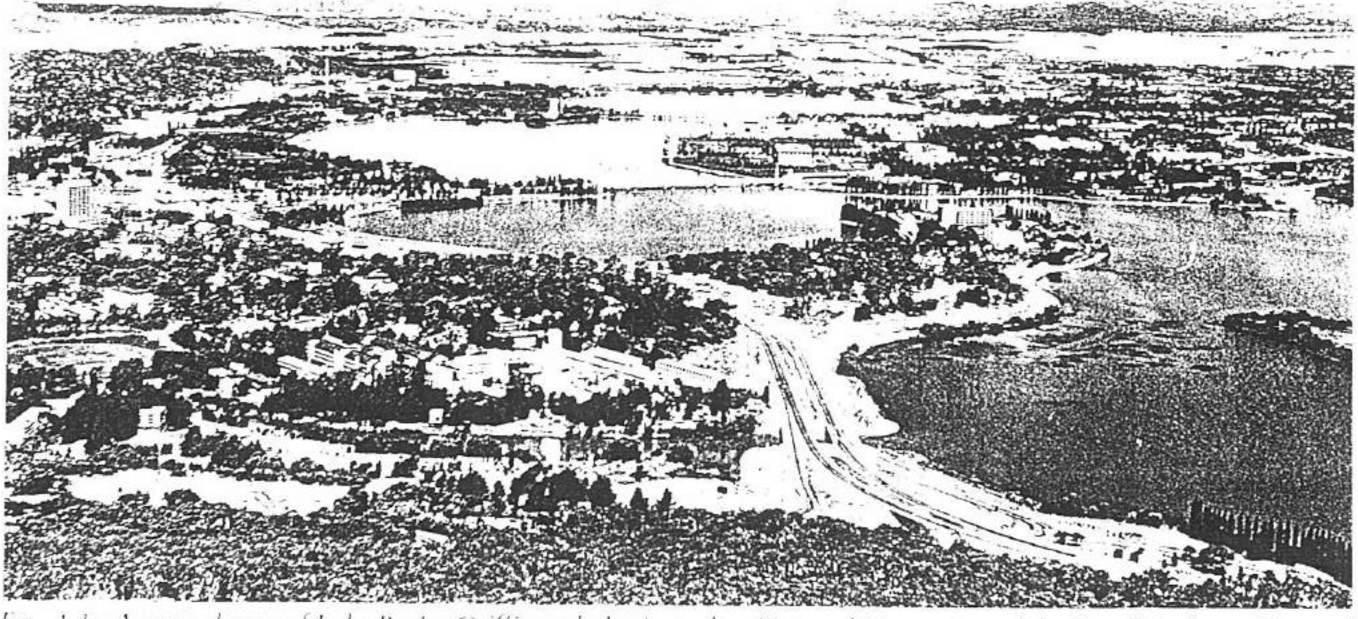


Fig. 4.1 A general view of Lake Burley Griffin with the Australian National University and the Royal Canberra Hospital in the foreground, and the Parliamentary Triangle and Russell Defence Offices in the middle distance.

and its parklands as the centre-piece of Canberra can now be seen and this, more than any other single feature, has led to the acceptance by the people of Australia of Canberra as their National Capital.

Lake Burley Griffin

Lake Burley Griffin is about 9 kilometres long, covers a surface area of 678 ha and varies in width from 300 to 1200 metres. It has about 33 kilometres of landscaped foreshores which provide access to 314 ha of parkland and 142 ha of the Eastlake Wetlands, a breeding ground for many species of water birds. It is a shallow lake with a maximum depth of almost 18 metres near Scrivener Dam and a mean depth of nearly 4 metres.

The lake evolved out of the investigations and debates of earlier years combined with the opportunities to adapt rapid advances in technology in a favourable political climate. The National Capital Development Commission, under the leadership of Commissioner John Overall, recognising the well of political and public support for the development of Canberra in the late 1950s, tackled forcefully the problems remaining from earlier years. The application of new and sophisticated techniques for dam and gate design for flood control, and of intensive hydrological activities led to construction of a dam across the Molonglo River below Black Mountain in 1963 and the filling of the Lake in 1964.

Completion of the physical works is not the end of the story because the maturing of the surrounding landscape in which 55,000 trees were planted and the introduction of beaches, picnic areas and other facilities around the lake shore is a continuing process which will delight the generations to come. The lake and its landscape lie at the heart of the National Capital but are also part of an open space system which provides a variety of recreational experiences for residents and visitors to Canberra.

The story of the lake's construction is one of vision and short-sightedness, of confidence and doubts, and of procrastination and performance out of which emerged a water feature consistent with the vision of Scrivener and the intentions of Walter Burley Griffin.

When the conditions for the competition for the design of the Capital were announced in 1911, they were accompanied by a more detailed survey on which Scrivener had shown the level reached by a flood in 1891. This and other information prompted most competitors to include a water feature in their designs.

An engineer, J.A. Smith, was one of the majority of judges who awarded first prize to Walter Burley Griffin in 1912 for his entry in the Federal Capital Design Competition. Their decision was subsequently upheld by Mr King O'Malley, the Minister for Home Affairs. Griffin had placed the central basin of his lake scheme across the land axis of his design, with two formal basins at each end forming his 'water axis'. Around these three basins and on these two axes, his main civic design compositions were arranged. Not content with this limited area, he submitted another plan 'rendered on cambric in monotone', to indicate the dominant topographical features and their relationship to the proposed architectural and landscape development.

This shows the irregular 'West Lake' at the same level as the formal basins, 556m and the balancing 'East Lake' set six metres higher. This was certainly the grandest scheme submitted, yet it had an appealing simplicity and clarity.

Griffin was a man with remarkable powers of imagination and a genius of topography. Unable to visit Australia, he studied a plaster model of the city site to a scale of about 1:5000, provided for the information of competitors in the British Consulate General in Chicago. From this, he had grasped, as his rivals and critics had not, the significance of the Molonglo flood plain.

The basic fact was that right across the middle of the city site lay a belt of land, averaging 0.8 km in width, which, despite the Competition conditions' promise of a regulating weir at least 23 km above the City, would always be in danger of flooding. It could not, except with great difficulty and expense, be built upon.

Two alternatives were possible, either the flood plain could be treated as a continuous park bordering a shallow stream, designed to suffer periodical submersion without damage, or it could be permanently flooded by damming the river at a point below the City, thus forming a chain of natural lakes.

As to which of these alternatives would be more effective in uniting the two halves of the city in a scenically dramatic way, Griffin was in no doubt.

He wrote in his competition report:

"The main waterway, the Molonglo, is left in its present state in the lowest and widest regions" ie., below the City . . .

". . . Next above and at the second of the weir sites suggested in the invitation program (i.e. at Yarralumla) a dam of very modest proportions, constructed in connection with one of the roadway crossings, floods the lower outlying informal lake (i.e., the West Lake) and the triple internal architectural basins which bound on three sides the government group for the reflection of its buildings, and for improvement of humidity conditions in the heart of the City . . . The most difficult problem connected with the waterway through the centre of the site is to minimise its interference with traffic and at the same time least cut up areas."

"The circular pools (ie., the East and West Basins) and the connecting (Central) basin provide three water bodies, each complete in itself, located in the spaces between the direct lines of communications from centre to centre. At the same time, because of their largeness of scale and severe simplicity, they conform to the architectural character of the centre of the City with its monumental groups and throngs of busy people."

Although awarded first prize by the Minister, Griffin's design was referred, along with other premiated designs, to a Departmental Board of experts for advice. The Board, on which Scrivener served, produced a scheme of its own, which contained another lake scheme, a near relation of the entry submitted in the design competition by the Australian group of Scott, Griffiths, Coulter and Caswell.

When construction of the capital was inaugurated on 20 February 1913, the Board's scheme was the basis for the City's development. Griffin subsequently was appointed 'Federal Capital Director of Design and Construction' on 18 October 1913. He then published his Preliminary Plan which shows the modifications resulting from his examinations of the site and his discussions with the Board.

He further refined his plan producing a "Schematic" Plan two years later. This was examined by the Parliamentary Works Committee in their enquiry into the Provision of Dams for Ornamental Waters in 1916. This forced Griffin to defend his scheme against the criticism of the former members of the Departmental Board and others. Scrivener, for instance, said "We would not agree with Mr Griffin. One of the points of contention being the form of the lake. I regard the artificial form as much less beautiful than the natural contour. It gets rid of the bays and indentations that are the principal charms of Sydney Harbour".

Griffin was unshaken in his belief in the formal elements of his scheme, defending it vigorously from all attack. He

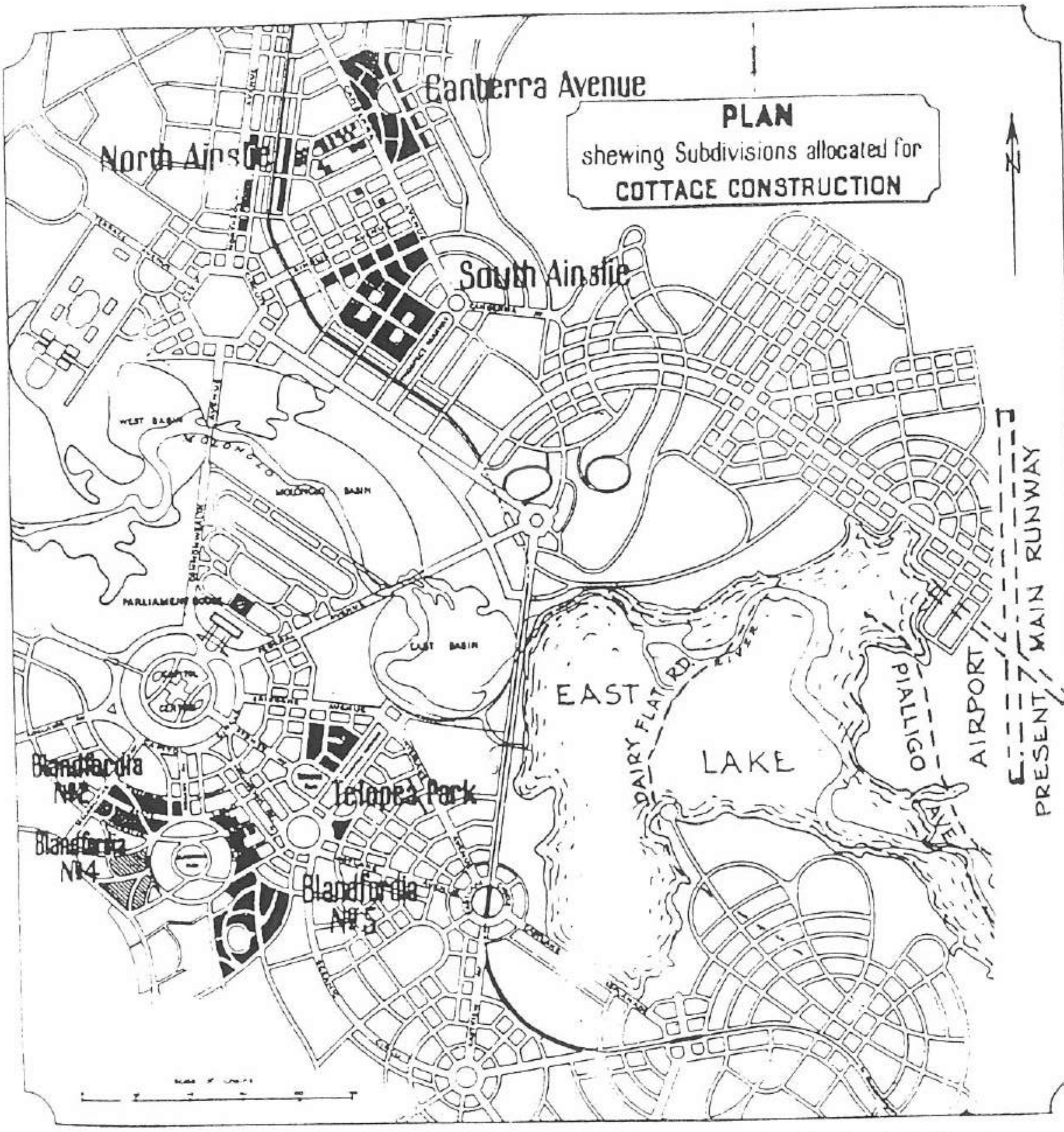


Fig. 4.2: A Griffin plan with East Lake added to show its relation to the present Dairy Flat Road, Pialligo Avenue and the Airport. This widespread East Lake was to be 6 metres higher than the present Lake Burley Griffin.

worked out schemes for the treatment of the formal boulevards that were to surround them, which he claimed were 'one of the reasons d'etre of the ornamental waters'.

In spite of Griffin's impressive stand, the Committee decided that the formation of East Lake should be indefinitely postponed, and that the shape of the formal basins should be modified, decisions which have persisted to the present day.

Most of the construction work that had occurred up to 1916 had been outside the City Area and therefore beyond Griffin's theoretical control, such as the dam on the Cotter River for the city's water supply which was completed in 1915.

Although Griffin's revised plan of 1918, with a few amendments, became the official plan for the National Capital following the passage of the Seat of Government (Administration) Act in 1924, there was a number of significant alternative, though interim, lake proposals considered over the years. In the main they consisted of schemes which would allow the progressive development of the full proposal and consisted of a number of weirs established to provide "a ribbon of water" between Yarralumla and the Causeway. The more significant of these were referred to Parliamentary Standing Committees on Public Works whose reports provide enlightening reading and an insight into the difficulties of those earlier years in assessing the feasibility of measures for flood and drought provisions.

The Owen and Peake Report prepared in 1929 is representative of the thinking of this intermediate period and its conclusions drew attention to several hydrological issues which greatly influenced the decisions on the size and security of storages. These issues were examined in more recent studies and with the support of more extensive data and research were able to be resolved. Thus the original concept proved practicable. A third ribbon of water scheme was suggested to replace the West Lake which had been incorporated in the Canberra Plan in 1933. The Wilson Report of 1955 brought the full proposals back

into line and with the subsequent appraisals of 1958-1964 led to the present lake.

These earlier schemes were summarised in a memorandum of April 1956 to the Parliamentary Standing Committee on Public Works (Metric equivalents have been substituted):

> The Owen and Peake Report of 1929, discussed the first Ribbon of Water Scheme suggested in 1926, but not approved by the Public Works Committee. That entailed a weir at Yarralumla at 548m level, and was to bank up the water only as far back as Commonwealth Bridge.
>
> The Owen and Peake Report suggested that, until the lake scheme was implemented, some of the objections to the 1926 Ribbon Scheme could be met by adding to the Yarralumla weir another small weir, at 551 level, near Scott's Crossing, to back up the water to the weir already constructed to the 553 level at the power house. This in turn backs up the water to the Causeway — the beginning of the former East Lake. This second ribbon scheme would therefore have made use of the main Yarralumla dam, the Scott's Crossing weir, and the power house weir to provide a continuous ribbon of water through the city. This scheme was not approved. It would have been relatively inexpensive, but depended entirely on the assumption that large control dams could be built on the Upper Queanbeyan River to provide water for city parks, etc., and sewerage dilution, as well as for flood control.
>
> Otherwise there would have been risk to the Commonwealth Bridge in flood times, through backing up by the Yarralumla weir.
>
> The third ribbon of water scheme, substituted for the west lake on the Canberra Plan of 1953, was a different proposal altogether. It aimed at placing a low weir at Yarralumla, and also a large dam at Lennox Crossing to form three main lake basins, and to use the area surrounding the ribbon for special gardens and recreation areas. This scheme would be enormously expensive — much greater than the lakes scheme — and, making no provision for flood control, would have been subject to frequent floodings. It was subsequently shown that the foundations for that weir at Lennox Crossing were most doubtful in that position, and a dam on the upper reaches of the Queanbeyan River for flood control would be impractical. A tremendous amount of water would be needed, covering a vast area of good country, and even then it would only delay the peak of the floods for a few hours.
>
> It was stressed in the Owen and Peake Report that because of lack of data their conclusions were not definite.
>
> The Wilson Report was made after a very careful survey of the area and all the records, which are now a deal more complete than in the time of the Owen and Peake Report. Mr Wilson based his findings on a somewhat different basis to the former report, but has left no ambiguity about the aims of it and the probable results. He makes it plain that, in drought years the lakes could fall by as much as 0.84m, but the occasions will be very few and in 50 per cent of the years there will be no fall at all. It is shown that the lakes scheme can be successfully implemented with those limitations, but a dam on the Upper Queanbeyan would be essential if some of the other requisites, such as flood control efforts, were to be insisted upon.
>
> The Wilson Report made no attempt to provide water for Sewerage dilution, as the amount required is now so great that the present river flow would not cope with it, and other measures will be required. He concluded with the suggestions that the lakes scheme should be implemented at the 556 level, and the disadvantages of it accepted for the time being. In the unlikely event of them proving really objectionable, it will still be possible to construct the smaller of the dams suggested on the Queanbeyan River to supplement the flow occasionally.
>
> The point to be remembered is that really, effective flood control would be impossible and the ribbon scheme from that aspect alone is most undesirable, but it is essential to carry on immediately with planning and preparatory works on the lakes, gardens, and bridges, and this matter must be determined without delay.
>
> The question of the aesthetics of retaining west lake in the scheme was doubted by the chairman of the Planning Committee, but a large majority of the witnesses in the Senate Committee's Inquiry were in favour of it.
>
> Mr Wilson's conclusion that the lakes scheme should be implemented at present without the dam on the Queanbeyan River for drought and flood control, was made with the full knowledge of the Owen and Peake Report. That report showed that even with the big dam at Googong, the 1925 flood would have been controlled for only 12½ hours.

The Senate Inquiry of 1955 led to the establishment in 1957 of the National Capital Development Commission which quickly recognised the importance of the lake. It was able to draw on the earlier studies and on the technical resources and hydrological data available through Commonwealth departments and authorities. NCDC

Fig. 4.3: Sailing on Lake Burley Griffin. Photo — NCDC.

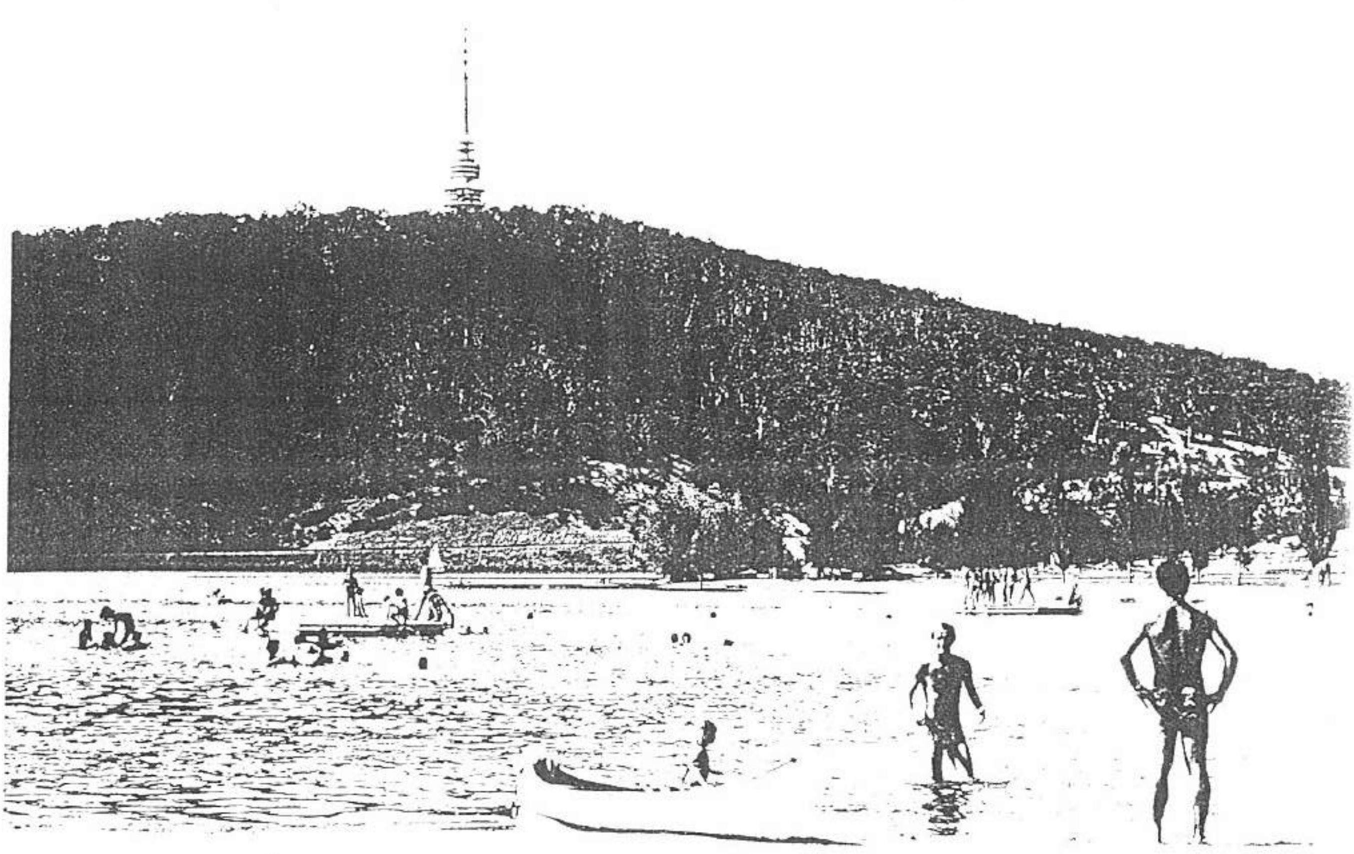


Fig. 4.4: Swimming in the lake is permitted everywhere except in the Central Basin. This is the beach at Black Mt Peninsula. Photo — Pieter Arriens for NCDC

studies led to a greater assurance on such issues as the behaviour of the lake in terms of floods and droughts and of scour and siltation. Other studies arranged by NCDC were able to provide satisfactory answers on water quality, effects of climate and health, hazards of unsightly margins, of mosquitos and midges and the possible disbenefits from changes in land uses.

A number of technical papers listed at the end of this chapter provide further detail on the investigations and designs for the Scrivenor Dam and Lake Burley Griffin. It is useful however to record a few interesting points relating to those issues which had plagued the earlier investigations.

Of all the investigations carried out, the hydrological studies were by far the most significant. The best use had to be made of the limited information available on weather and river flows. The question of the availability of water with or without an upstream storage and the effect of future flows, particularly in the Parliamentary Triangle, could not remain unresolved.

The Molonglo River which feeds the ornamental lake in Canberra with an average annual inflow of 180 cumecs has three main tributaries which rise to the east, south-east and south of the City. The catchment with an area of 1810 km^2 is subject to the spillover from heavy coastal storms which have, in recent years, produced flood peaks up to 3,540 cumecs (cubic metres per second).

Following the detailed theoretical analysis of rainfall patterns and river flows,[5] a river model was constructed which was used to test the adequacy of the theoretical findings. It was shown that these theoretical calculations were extremely accurate and the verification obtained from the model studies was most reassuring and enabled far more detailed information to be provided on many aspects. The usefulness of the main model studies led in later stages of the design to the development of more specific model studies, and, in all, some four models were constructed. The main river model also served as a useful medium for informing those in authority and the public at large of the implications of the lake scheme.

Some of the detailed investigations carried out on the models related to the alignments of the lake shore, the positioning of the bridges, the details of flood levels and the behaviour of the flood gates. For example, Kings Avenue Bridge was resited about 50 metres north of its original proposed location increasing the useful waterway from 60 to 80 per cent of available area. The studies also confirmed that with the proposed gates the lake level of 556 m could be maintained in the central areas for all floods up to 2,300 cumecs and that at the design discharge of 5,600 cumecs the level of the lakes in the central area would not exceed 560 m and that such levels would be controlled not by the lake structure, but by the bar of Black Mountain Peninsula.

The use of the models also allowed studies to be made of the shore alignment, the design of the energy dissipator, the handling of floods through the East Lake area and the potential benefits to be achieved by realigning the main channel leading to the creation of interesting islands.

At the other end of the hydrological scale careful studies were made of drought conditions over past years and allowances made for evaporation, irrigation and leakage. It was determined that the lake would function quite satisfactorily within a metre of the 556 m water level without an upstream storage. By this time the application of British Standards for dilution of sewerage effluent was no longer relevant or practicable.

In addition to general ecological studies, specific investigations related to fogs, fish, the behaviour of aquatic plants, the likely extent and magnitude of waves, conditions required to prevent breeding of mosquitoes and other insects, and matters relating to the use of the lake for a wide range of recreation activities. These were carried out to confirm the feasibility and establish the basic criteria for the design of the lake itself.

The investigations extended in this way over the matters of geology, the expected rise in the water table, the quality of the water, the effect of upstream operations, including discharge of effluents into the Molonglo River and matters of turbidity, sedimentation and erosion.

This latter field of study probably gave the greatest concern because the lake was to be a relatively small, shallow body of water downstream of a large catchment subject to very high flood discharges, capable of carrying considerable quantities of sediments. Hydrological science at the time did not offer a reasonable method of estimating the proportion of such sediments that would be trapped by the lake.

After extensive studies using a number of highly respected advisers, it was determined that provided satisfactory precautions were taken in the catchment, the lake could be expected to function reasonably satisfactorily. Nevertheless, some floods could deposit substantial quantities of sediments, particularly in the upper reaches of the lake. It was impracticable to carry out quantitative studies of sedimentation in the lake and the design has therefore endeavoured to make conditions as favourable as possible for minimum sedimentation.

Under small floods with only one gate down at the dam, an opening 32 metres wide by 5.2 metres high can allow large quantities of sediments to pass straight through the lake. Under any floods above 2,300 cumecs five such gates would be down.

Bed load traps were provided upstream of the dam and widespread soil conservation measures were carried out throughout the ACT portion of the catchment. In addition, an agreement was made with the State Government for a large soil conservation programme to be undertaken in the much greater NSW portion of the catchment. Much of this programme was well underway by the time the lake was built. Such programmes of course make major improvements to property values and hence landholders paid one third of the costs with the Federal and State Governments sharing the remainder.

Scrivener dam consists of a concrete gravity section with five 32 metre x five metre flap gates between two concrete gravity buttress non-overflow sections of 71 metre total length, and of earth embankments 184 metres long of which a total length of 40 metres has a centre concrete cut-off wall founded on rock and one metre thick. Maximum structural height of the dam is 36 metres.

Fig. 4.5: Minister for the Interior, J.D. Anthony, invites the Prime Minister Robert Menzies, to inaugurate Lake Burley Griffin in 1964. Photo — NCDC.

The dam is founded on quartz porphory which was covered by alluvium of varying thickness. On exposure of the foundation under the river itself, a combination of geological faulting required the use of post tensioned cables to tie several blocks of the dam back to the sound rock upstream.

A roadway is provided across the dam to serve as a river crossing between Woden and the City and Belconnen. It is also used as a means of gaining access for the maintenance of the gates.

The size of the design flood, that is the flood which had to be handled by the structures in the flood plain was determined at 5,600 cumecs. The structures were also examined for a flood of 8,500 cumecs in terms of any possible catastrophic damage arising from this 'max max' flood situation.

In calling tenders for the design, manufacture and erection of the five 32-metre crest gates to pass such floods, it was necessary to ensure the minimum obstruction to the passage of debris. The neatness and appearance of the gates also was considered of great importance. The gates as erected are fish belly flap gates designed by Rheinstahl Union Bruckenbau, West Germany. The water load is carried by the steel skin plate of the fish belly section to six cross beams. Each of these cross beams is supported by a hinge, anchored to the concrete dam crest and the four centre beams are also supported approximately at their half points by hydraulic jacks.

The main criteria in the control of Lake Burley Griffin is to keep the water level in the Parliamentary Triangle as near to 556m as is possible with the gates provided in the dam. Three one metre × one metre sluice gates have been installed and can automatically adjust the outflow from the lake for a range of 150mm change in water level.

By setting the float-operated control equipment for the first sluice gate at slightly below the desired Top Water Level, the mean annual inflow of 5.1 cumecs will raise the water level to RL556. Under steady flow conditions the sluice gates can pass a discharge of approximately 60 cumecs without allowing the water to rise above 556. During periods of minor floods the filling of the lake storage above 556m is expected to enable the sluice gates to handle floods with peak discharges of less than 100 cumecs. In some years it will not be necessary to use the flap gates for flood discharge at all. The longest recorded period that flap-gate operation would not have been required was from December 1925 to March 1929.

The first contract (for the gates) was let in May 1962, work commenced on the Dam in September 1960 and the storage commenced to fill in September 1963. Apart from the unfortunate combination of faulting encountered in the foundations and from normal troubles experienced in the installation of such large gates, the construction of the Dam proceeded well.

The treatment of the lake margins [5] varies according to their location and has regard to function, hydraulics, cost, maintenance and the landscape value of particular designs. The interest and beauty of the lake arises as much from the variety in the 33km of shoreline and its treatment as from the area of water itself. Apart from the formal south bank of the central basin, the shoreline is quite informal and seeks to provide this interest. There are four main types of margins.

- A concrete wall consisting essentially of a low reinforced concrete retaining wall capped by a coping is provided in the formal sections of the lake, particularly where hydraulic conditions require such treatment and the foundations lend themselves to this type of wall. It is designed to allow a fall in lake level without an exposure of the lake bed. The precast coping is capable of adjustment and has enabled a most satisfactory line to be achieved on the long straight margin.
- In other areas a grouted rock wall has been provided and is an effective treatment from the point of view of maintenance and freedom from erosion and has been extensively used in the upper reaches of the lake where the burden of the incoming floods have to be withstood.
- The third form of edge treatment was the provision of sand and gravel beaches which are designed to allow some protection to the subsoil and as a provision for entry to the lake for recreational purposes.
- The fourth type of margin is essentially a natural margin, where there are rock outcrops and steeply sloping stable shores. The western areas, in particular, have extensive sections of such foreshores which have been planted for landscape and stability purposes.

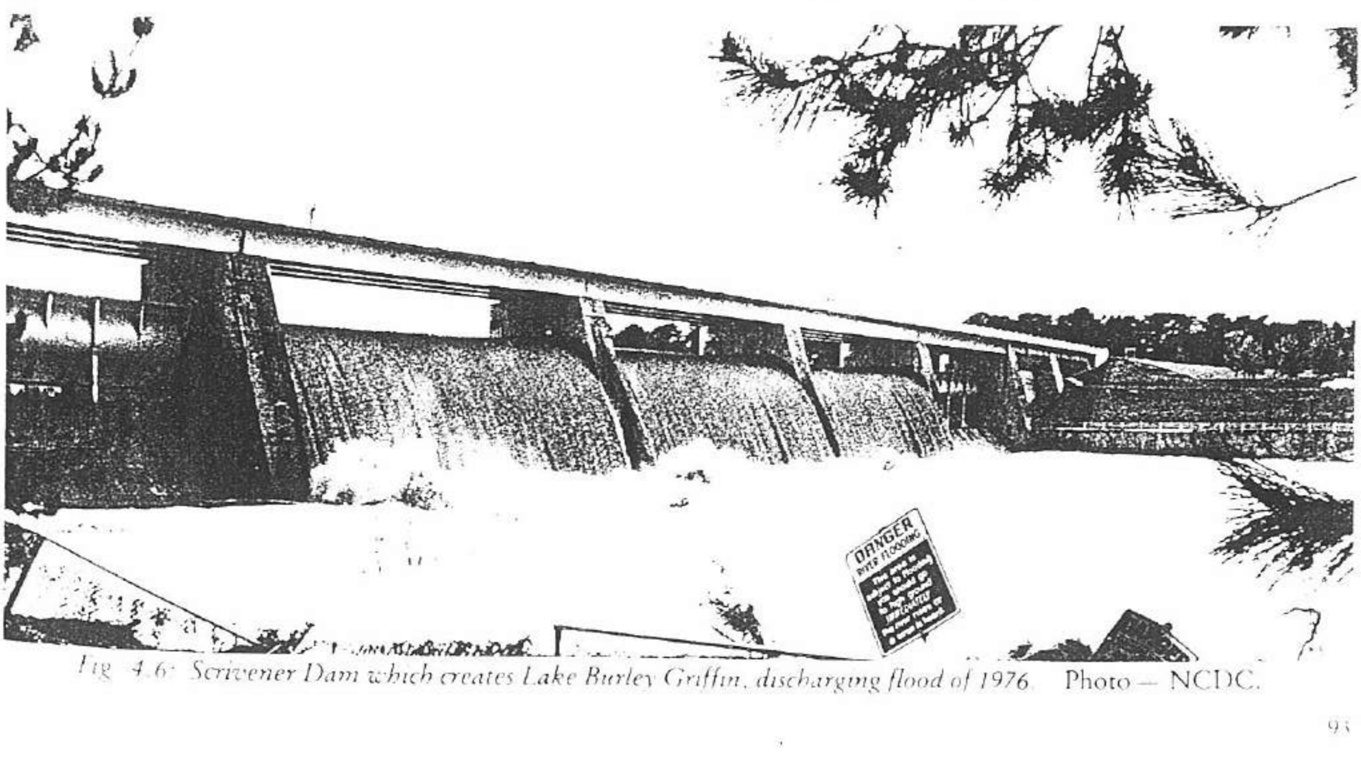


Fig. 4.6: Scrivener Dam which creates Lake Burley Griffin, discharging flood of 1976. Photo — NCDC.

The location of the lake margins generally follows the 556m contour which proved an extremely economical location. There were several areas of shallow depth or special functional requirements which have been developed to provide particular features in the lake. For example, in the West Lake area near the University, cut and filling resulted in a larger lake and an interesting island. Similarly, a balanced programme of earthworks led to the developments of the boat harbour in East Basin, the Nerang Pool in an area which was formerly swamp land and the formation of Yarralumla Bay and Lotus Bay for boat shelter. To allow a larger triangular sailing course for races, the 'finger' of land at Yarralumla Bay was cut out but the 'finger nail' left was another island (Spinnaker). The lake has six islands altogether.

Any reference to Lake Burley Griffin would not be complete without a statement on the design and construction of the traffic bridges which contribute so much to the total composition of the central areas. These are discussed in Chapter One.

Thus it was with the completion of engineering works in September 1963 that all that was required was a supply of water — something beyond the powers of Prime Minister Menzies, Commissioner Overall or the many highly skilled and enthusiastic professionals who had contributed so much to the lake's construction. As the dry season which had so favoured construction operations continued, doubts began to emerge that the National Rowing Championships, scheduled with an abundance of rowers faith for Lake Burley Griffin 2 May 1964, would become a modified version of the Todd River Regatta. An alternative course was being prepared for use on a partly filled lake when at the end of April 1964 heavy rains fell on the catchments, the lake filled and the Regatta was held successfully, though in the midst of some flotsum and jetsam from the receding flood.

In the years since then, the lake and its parklands have proved universally popular. Power boats are not permitted, other than for safety patrols and the coaching of rowing crews. However on most Saturdays and Sundays in summer, more than 10,000 people are attracted to the lakeshore, more than one-third arriving by car at the one time. The increasing usage of the lake and its foreshores is generating a demand for more beaches and the provision of further facilities associated with direct use of the lake, such as boat sheds, clubhouses, boat launching areas, parking and other structures and amenities.

The pressure of people in some areas leads to conflicts between different kinds of uses and users that need to be resolved by management. There is also increasing demand for sites for tourist oriented development. As the lake and the foreshore are a finite resource, it is desirable that planned uses and facilities are located in accord with its physical character and environmental capability and that the management implications of this are recognised.

Swimming in the lake is only prohibited in the formal Parliamentary area but water quality problems have occasionally caused closure of the lake for short periods. Despite the difficulties in controlling the quality of all inflows, the lake water quality remains mostly acceptable by swimming water quality criteria.

Continuing careful attention to all aspects of lake management is vital, particularly as the lake matures.

Lake Ginninderra

It is not surprising that following the impact of Lake Burley Griffin on the Canberra scene, the availability of water and its maximum beneficial use became an important factor in the development of Canberra's new towns. There was in the mid 1960s a sensitivity to environmental matters, an awareness of the need to preserve water quality and a changing lifestyle which placed greater emphasis on social and recreational matters.

In the early planning considerations for the new town of Belconnen, the pattern of neighbourhoods and regions focusing on the town centre defined the general location for that centre. In the detailed consideration of the town it was realised that an opportunity existed, subject to detail

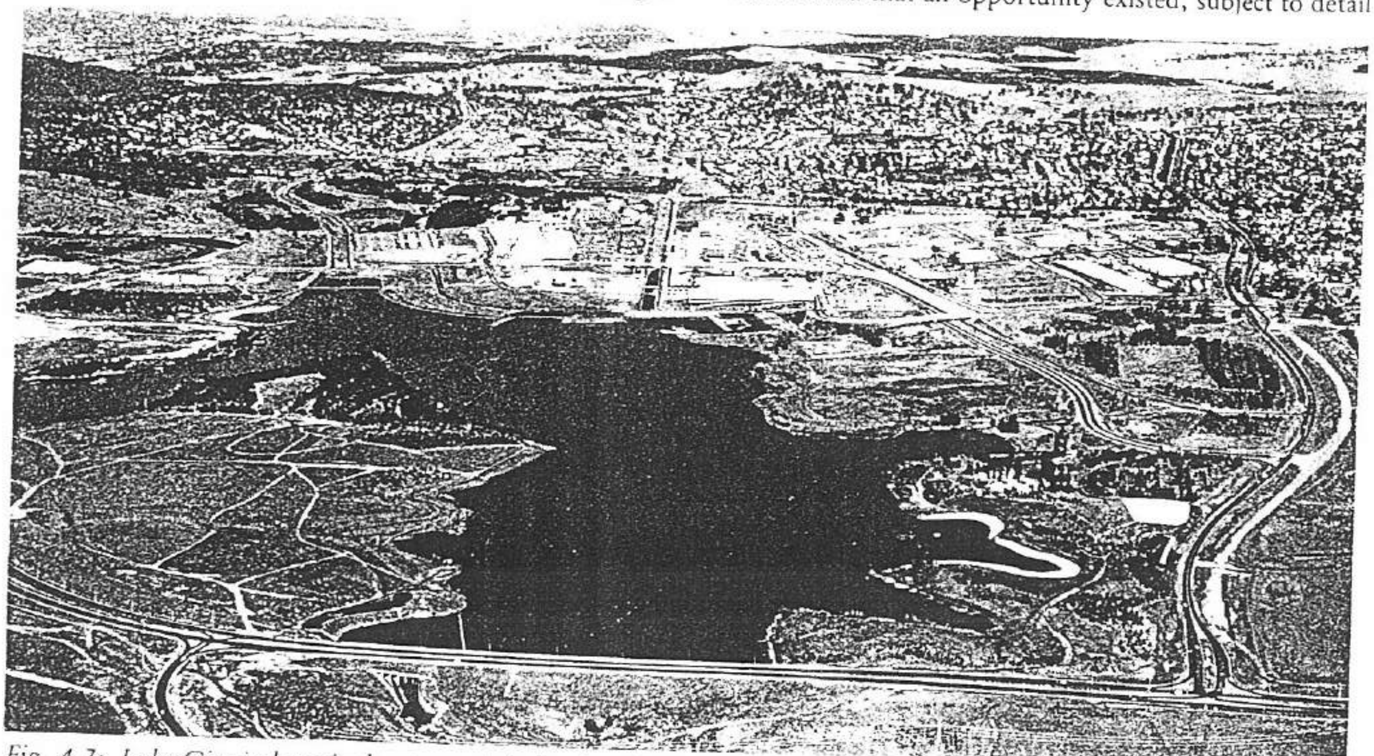


Fig. 4.7: Lake Ginninderra is about one-sixth the size of Lake Burley Griffin. Belconnen Town Centre is being developed on its southern foreshores. Photo — NCDC.

River Model, Spillway Model and Dam Design for the Canberra Lake Scheme

BY

A. J. CONDON, B.E., (*Graduate*) B. V. KEARSLEY, B.E. (*Associate Member*) and A. FOKKEMA (*Associate Member*)*

Summary.—This paper presents some aspects of the design and operation of Lake Burley Griffin carried out by the Commonwealth Department of Works on behalf of the National Capital Development Commission. Part I describes a fixed bed distorted river model used to determine the behaviour of the design flood. Part II describes the development of a suitable energy dissipator using a spillway model. Part III outlines some aspects of the design of the gravity dam and the operation of the sluice and crest gates.

PART I

Hydraulic River Model Studies for Lake Burley Griffin

BY A. J. CONDON

1.—Introduction.

The Molonglo River which feeds the ornamental lake in Canberra with an average annual inflow of 180 cusecs has three main tributaries which rise to the east, south-east and south of the city. The catchment with an area of 750 square miles is subject to the spillover from heavy coastal storms which have, in recent years, produced flood peaks up to 125,000 cusecs. The purpose of these model studies was to assess the design requirements of the lake structures when these sharp peaked floods are routed through the lake area.

Although extensive computations of the design flood backwater had already been carried out it was felt that a fixed bed river model could be used more explicitly to:

(*a*) Check and extend the computed backwater curves for a range of flood flows.

(*b*) Determine in conjunction with the spillway model, described in Part II of this paper, the size and operating characteristics of the gated dam structure.

(*c*) Determine areas of possible scouring and areas of deadwater prone to siltation.

(*d*) Investigate suitable realignments of the edges and channels to improve the efficiency of the lake to discharge a flood wave.

(*e*) Position the two high level bridges and the low level causeway at their most efficient locations (see Fig. 1).

(*f*) Determine the approach and tailwater conditions of the dam at various flows for use in the development of the spillway.

Further it was felt that the model would have a large publicity and demonstration value.

2.—Topography and Hydrology.

The river channel at the head of the lakes has a bed slope of about 1 in 1,000 and a flow capacity for just over 8,000 cusecs. This channel meanders through a well defined grassed flood plain varying in width from 1,500 to 5,000 ft. which is completely inundated by a flood of 20,000 cusecs and allows no appreciable additional spread of water for floods up to 300,000 cusecs. The limits of this flood plain are generally the limits of the lakes as shown in Fig. 1. For the two miles upstream of the dam, the river narrows to a more tortuous section with a slope of 1 in 700 and has no flood plain. Throughout its length the river channel is thickly laden with willow tree growth.

Structures on the lake are to accommodate a maximum design flood of 200,000 cusecs and withstand without major catastrophe a flow of 300,000 cusecs. The expected probability of flood recurrence developed from records on the Molonglo and neighbouring catchments is shown in Fig. 2.

3.—Description of Model.

3.1 Design:

To ensure sufficient approach conditions to Kings Avenue Bridge and to establish tailwater levels for the dam, the model extended some distance upstream and downstream of the lake area. The available space and the desire to keep the model as large as possible established a horizontal scale of 1 : 400. Considerations of viscous effects, range of required flows, accuracy of measurements and variation in roughness established as the most favourable a distortion of 1 : 5 giving a vertical scale of 1 : 80. With this distortion, a change in flow from 100,000 cusecs to 200,000 cusecs was calculated to induce an error of only about 4 per cent in the head loss due to textural roughness (Ref. 1).

Similarity was established on a Froudian relationship $\left(F = \sqrt{\frac{V^2}{gL}}\right)$ and the velocity scale of 1 : 9 and discharged scale of 1 : 286,000 followed.

The overall size of the resulting model was 110 ft. by 25 ft.

3.2 Construction:

Closely spaced vertical masonite templates, cut from a one-foot contour, were used to mould the cement mortar surface over a weak ash concrete base. These templates were positioned using a grid control from a datum rail mounted on a surrounding brick wall. Spot checks over the completed model showed that modelling errors were very small. Water, recirculated from storage tanks at the downstream end, was regulated by a 6-in. gate valve over a standard "V" notch weir before entering a baffled head tank. The tailwater was controlled by a flap gate over which the water passed to enter the storage tanks.

3.3 Instrumentation:

Stage heights were measured with a surveyor's level, mounted onto a rigid base, and a staff graduated into the vertical scale. Velocities were recorded using a micro-current meter with the signal boosted through a radio amplifier. When the depth of flow was too small for this meter, surface velocities were measured by timing small floats.

A series of piezometric tappings embedded into the river at various sections were recorded in boxes on the outside walls using converted micrometer screw gauges reading to 0.001 in. Surface flow patterns were recorded by photographic time exposures of confetti floats while sub-surface patterns were observed after injecting a fluorescein dye.

*This paper, No. 1800, was presented before the Engineering Conference, 1964, in Canberra from 13th to 17th April, 1964.

Mr. Condon and Mr. Kearsley are Engineers Class II, and Mr. Fokkema is Principal Hydraulic Structures Engineer, all of the Commonwealth Department of Works.

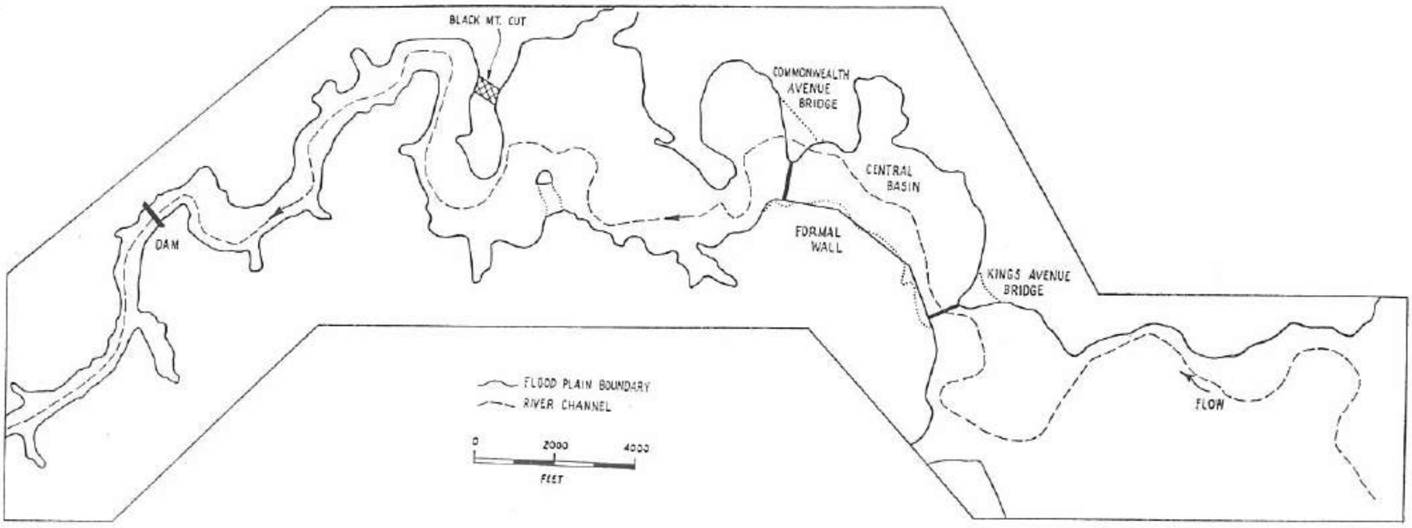


Fig. 1.—Layout of River Model.

3.4 *Operation :*

All floods on the model were carried out at constant discharge. The discharge was run for some time to ensure the base was fully saturated and that the inflow balanced the outflow.

4.—Verification.

The value of the studies depended upon the ability of the model to reproduce, with accuracy, the prototype flow conditions. Despite the 50-year intention of building the lake, information suitable for verification was available initially from only two floods. This was supplemented later by a flood which occurred in October, 1959.

(i) *27th May*, 1925.—This flood had a peak discharge estimated at between 110,000 and 160,000 cusecs. Accurate debris levels were recorded at about 500-ft. intervals along both banks throughout the modelled length and a series of photographs were recorded both during and after the flood.

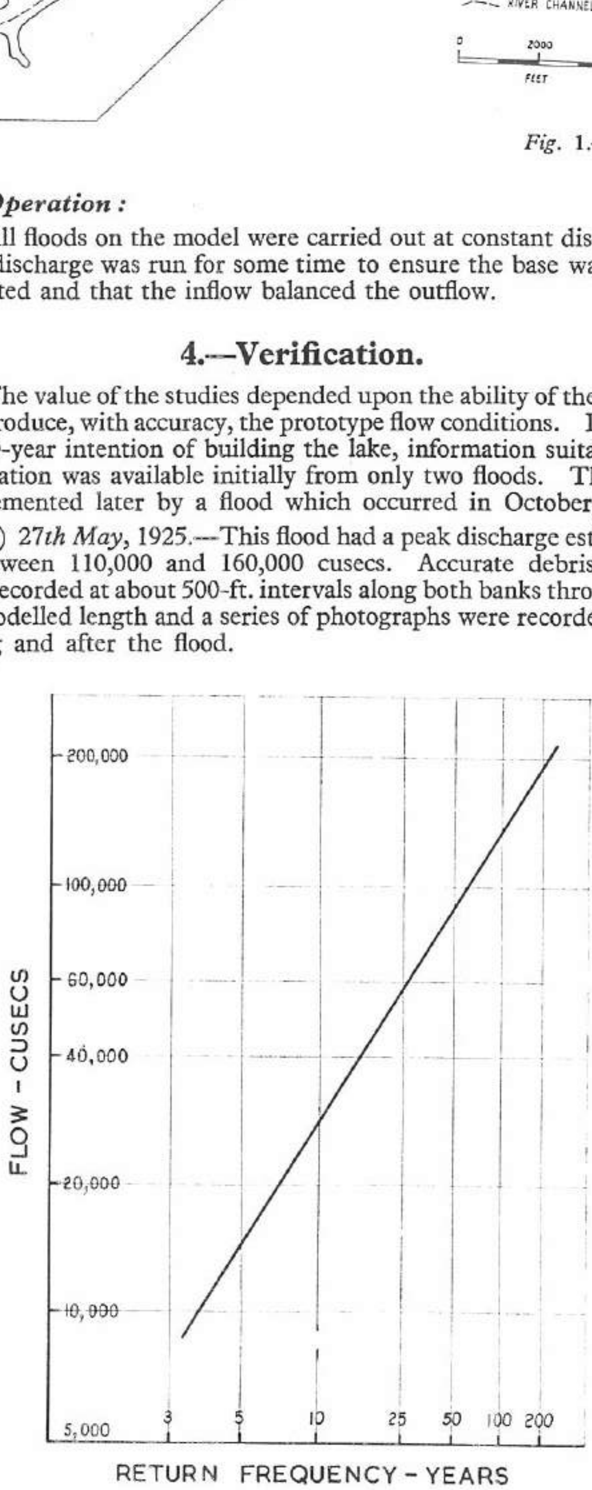


Fig. 2.

(ii) *25th June*, 1956.—A peak of 30,000 cusecs was gauged at the old Billabong-Commonwealth bridges crossing. The debris levels were recorded over the top half of the model, hydrographic recordings were made at several sections and a full aerial photographic coverage was taken.

(iii) *21st October*, 1959.—A sharp peaked flood of an estimated 50,000 cusecs. A discharge of 31,000 cusecs was gauged from the Billabong-Commonwealth bridges five hours after the peak. Weather conditions prevented the taking of aerial photographs but a sketchy cover was obtained from the ground. Debris levels were recorded over the extent of the model.

After some consideration of Mannings "n" and the equivalent grain size "k" the textural roughness of the entire model was increased by bonding a sieved gravel of 0.15 in. mean diameter to the surface with cement slurry. Willow tree growth was simulated in location and intensity using folded bird wire, and scale replicas of the bridge piers were placed into position.

With the 30,000 cusecs flood it was found necessary to supplement this roughness in local areas of heavy undergrowth, along fence lines and within the river channel. This additional roughening was achieved with fly wire cages containing varying amounts of $\frac{3}{8}$-in. gravel. When all debris levels were reproduced at this flow by adjusting the roughening the gauging was checked at the bridge openings. Agreement to within 10 per cent was received with the prototype velocities and to within 5 per cent of the total flow passing through each span opening.

Adjustments to the conditions of the 1925 flood were then made by removing a span (added after this flood) from Commonwealth bridge, reducing the supplementary roughening and matching the "tree" intensity to the photographic record. Several discharges were tried until it was found that the prototype flood slope was reproduced at 125,000 cusecs. Two points near the damsite disagreed with the model and were not consistent with any local features. A check on the original survey field books disclosed a level reduction error.

Before the 1959 flood occurred, the model had been altered extensively in the central areas to conform with the new proposals. This flood was reproduced, therefore, over the lower half only, but it covered that area not previously included in the 1956 flood. Tree growth was plotted directly from the prototype and areas of eddies, deadwater and local headloss observed in the flood were reproduced.

The reproduction throughout of all stage heights for each of the three floods, to within an accuracy of one foot prototype, was considered a very satisfactory verification. This agreement over a typical section of the river is shown in Fig. 3.

The comparison between the computed design flood backwater and that obtained from the model under the same conditions is

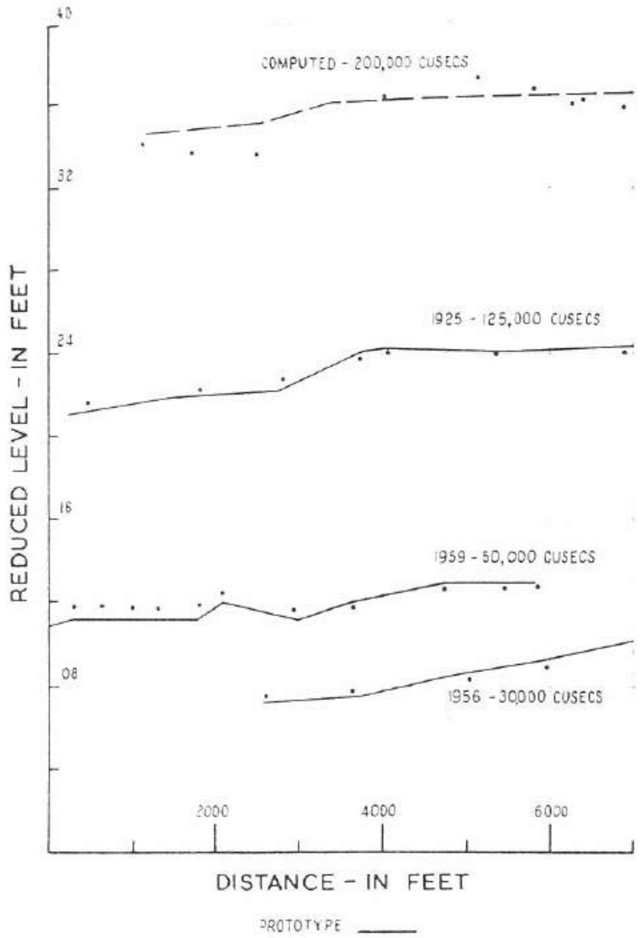


Fig. 3.—Results of Verification.

also shown in Fig. 3. The agreement between these two independent estimates lends confidence to them both.

5.—Tests and Results.

All tests were performed under " lake conditions ". Apart from the geometric reproportioning, the biggest adjustment made to the now verified roughness was the clearing of all tree growth in the heavily clustered river channel.

The proposals for the configuration of the embankments, foreshores and artificial islands were initially moulded in plasticine and finally in cement mortar as each proposal was confirmed.

5.1 *Bridges :*

By the time verification was complete, tenders had already been prepared for the seven-spanned twin-structured Kings Avenue Bridge providing a waterway area of 26,000 sq. ft. for the discharge of the 200,000 cusec design flood (see Fig. 4).

Preliminary tests indicated that in its tender siting the bridge would, at all major flows, suffer a gross asymmetrical flow pattern. Under the design discharge little more than 60 per cent of the waterway was effective, the flow was directed at angles in excess of 30° to the pier axes while velocities were recorded in excess of 16 ft./sec. Apart from the additional water loads on the piers concern was held for the erodible foundation materials consisting of highly weathered shales.

A series of tests, each recording head loss, flow pattern and velocity distribution, were pursued for twenty-seven different bridge and approach combinations. The selected arrangement involved a resiting 150 ft. north of the tender location. The extent of this move was restricted to the Avenue's radial axis and the position of Commonwealth Avenue bridge which is joined for aesthetics with Kings Avenue Bridge by a formal wall along the southern foreshore of the central basin.

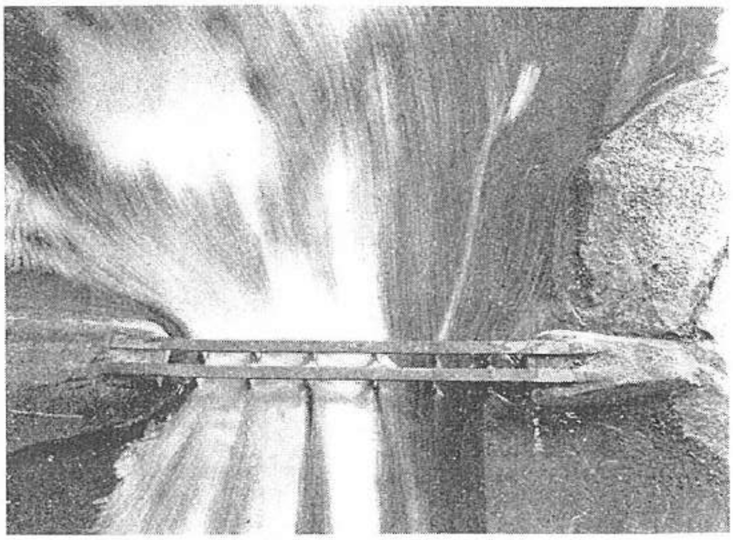


Fig. 4.—Kings Avenue Bridge—200,000 cusecs Discharge.

This shift, together with modifications to the approach embankments resulted in a velocity reduction to about 10 ft./sec. peak, an effective waterway of over 80 per cent of the area and a flow aligned at a maximum angle of 20° to the pier axis.

At Commonwealth Avenue Bridge, hydraulics and town-planning were more in harmony. A similar five-spanned twin-structured bridge had a waterway area of 32,000 sq. ft. for the design discharge and was located centrally and square to the natural flow path. An almost ideal pattern of velocity distribution with an 8 ft./sec. peak resulted.

5.2 *Floodgate Capacity and Backwater Curves :*

The design of the lakes required that the dam have a gate aperture such that under the 200,000 cusec flood the level of the central basins would not rise above R.L.1837 or 12 ft. above lake level.

As the cost of the gates increased rapidly with their height, they should be as long as possible. With the crest length limited to 500 ft. by the available spillway basin width, it was possible to establish, from the model, the total head level at the damsite to satisfy this condition. Accordingly tenders were called for the supply of five 100 ft. flap gates to discharge 200,000 cusecs at a total head of R.L.1831 at the dam.

Subsequently, using the spillway model and the gate manufacturer's rating curve, it became possible to reproduce a rating curve at any section throughout the lake or to plot the backwater curves for a range of flow conditions.

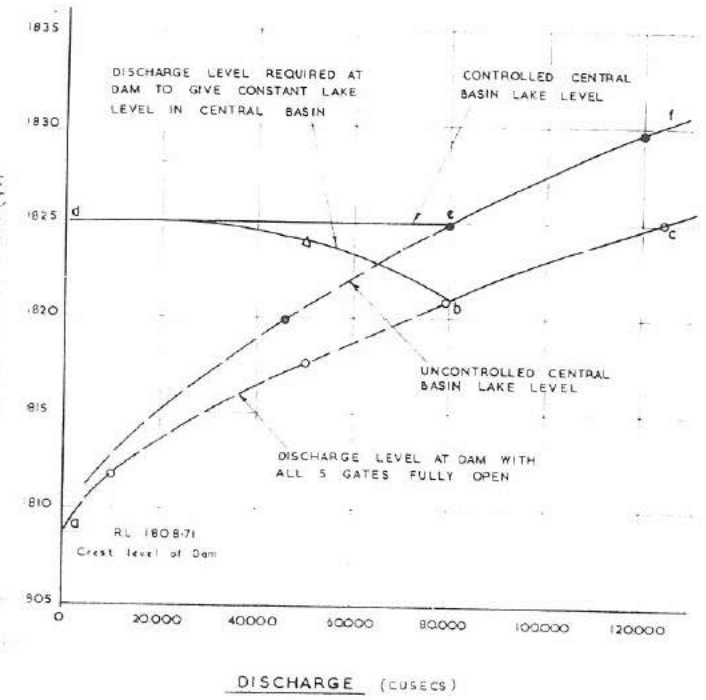
From Fig. 5 which shows the rating curves at the damsite and at Commonwealth Avenue Bridge it can be seen that for a total head level of R.L.1821 at the damsite the central basin can no longer be held at R.L.1825. This represents a flow of 80,000 cusecs which has a flood return frequency in excess of once in 45 years.

5.3 *Other Tests :*

An investigation was made on a proposal to reduce the flood levels by cutting through the Black Mountain Spur (see Fig. 1). It was found, however, that the cost of the one foot gained was far more expensive than the additional gate size required.

Flow patterns recorded to establish areas prone to rapid silting showed that very large areas of the lake will be deadwater even under the most adverse flood conditions.

The model was used to determine feasible piped service crossings and to supply tailwater and approach flow information to the adjacent spillway model and was used by the two firms of consulting engineers engaged on the design of the West Lake and Central Basins areas with respect to foreshore protection measure. It was also used extensively as a demonstration and publicity medium.



g. 5.—*Variation of Lake Level in Central Basin with Discharge Level at Dam.*

Conclusions.

The resiting of Kings Avenue Bridge 150 ft. north of its tender location increases the useful waterway from 60 per cent to 80 per cent of the available area.

Within the limits of the town planning, Commonwealth Avenue Bridge is located at the best hydraulic position available.

The lake level of R.L.1825 can be maintained in the central areas for all floods up to 80,000 cusecs.

The design discharge of 200,000 cusecs will raise the level of the lakes in the central areas by 12 ft.

Large areas within the lake will remain deadwater throughout extreme flood conditions.

PART II

Hydraulic Model Studies for the Canberra Lake Dam

BY B. V. KEARSLEY

1.—Introduction.

The purpose of these studies was the design of a suitable lling basin for the dam and, in conjunction with the River Model scribed in Part I of this paper, to derive basic rules for the eration of the spillway flood gates.

The main hydraulic features relevant to the design of the stilling asin and the gate operation were as follows.—

(*a*) The gated part of the dam extended over the flat slopes of the valley on both sides of the river. The stilling basin therefore had to be stepped and/or sloped transverse to the direction of flow, in order to minimise excavation and avoid having the stilling basin excavation below the dam foundations.

(*b*) Because of the different bay elevations in (*a*) above, there was insufficient tailwater depth in the higher bays at lower flows. Therefore, a gate opening sequence was necessary for the early and closing stages of large floods and the far more frequent low and medium sized floods were limited to the gates opposite the lower bays.

(*c*) During floods, the spillway flood gates had to be operated so as to control the lake levels some five river miles upstream in the Central Basin.

(*d*) The middle two gates opposite the lowest stilling basin bays were to be automatically controlled by switch gear operated from an upstream float well. They would normally open and close together.

(*e*) Normal low flows up to about 2,000 cusecs were to be passed through three 4-ft. x 4-ft. sluices through the body of the dam.

(*f*) The maximum design flood and the max. probable flood were 200,000 cusecs and 300,000 cusecs respectively.

The spillway and stilling basin layout finally adopted is shown in Fig. 1. Energy dissipation is by means of a series of hydraulic jumps in each bay assisted by chute blocks, baffle piers and an end sill.

2.—Description of Model.

2.1 Model Design and Construction:

The model linear scale was fixed at 1 : 72 (undistorted) and was mainly governed by space limitations in the laboratory. There is a sharp bend in the river just upstream of the dam, which, from observations on the river model, caused the approach velocity distribution to be severely asymmetric, and formed a large, slow moving eddy on the south bank near the dam. It was considered that these approach conditions could significantly affect the distribution of discharge between the spillway bays, and the bend was therefore included in the model.

The downstream section was relatively short as the tailwater rating information available was considered sufficiently accurate. Tailwater levels were therefore adjusted artificially by a sluice gate at the model outlet.

The model contours were formed by moulding weak, lightweight concrete made from mortar, boiler ash and clinkers to within 2 in. of the top of masonite templates and finishing with a mortar capping. The dam, gates and stilling basin were made of wood.

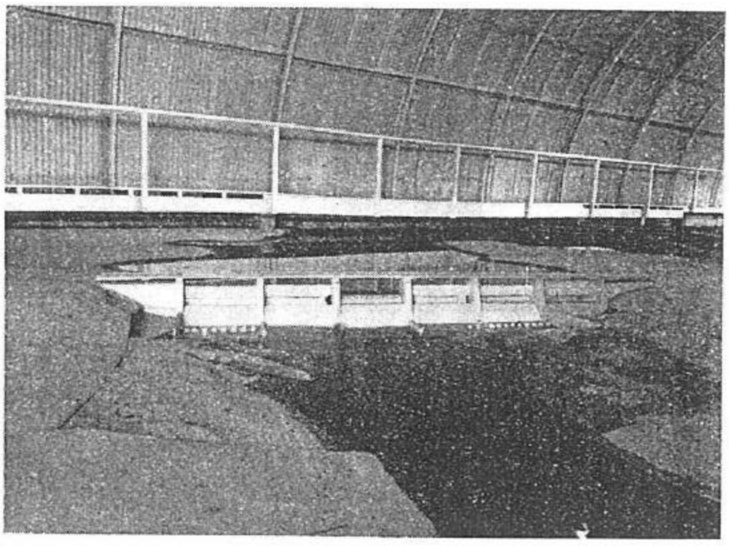


Fig. 6.—General View of Model.

A section of the river bed and valley sides immediately downstream of the stilling basin was filled with coarse sand of grain size representing 2 in. to 9 in. rubble on the prototype. The sand was moulded to the existing contours before each test, but quickly scoured into a stable pattern under the test flows. The scour pattern was used to compare the relative scouring tendency of each test arrangement (Ref. 4).

A general view of the model is given in Fig. 6.

2.2 Tailwater Control:

Actual flood levels at the dam site were available for the May, 1925 flood (125,000 cusecs) and the October, 1959 flood (50,000 cusecs). The river model studies over the river section downstream

of the dam showed that the 50,000 cusec flood levels were greatly dependent on the flow resistance due to the tree growth on the river banks. This was simulated on the river model by folded wire mesh as described in Part I of this paper. The 125,000 cusec flood was much less dependent on the trees, presumably because they occupied less of the greater flow cross section. As the tree growth could be variable over the years, or even be removed, water levels were obtained from the river model with the simulated tree growth removed. These levels were only dependent on the river bed levels, alignment and form, and were thus considered sufficiently permanent to use for the basic design of the stilling basin. They are shown plotted in Fig. 7 with the available prototype data.

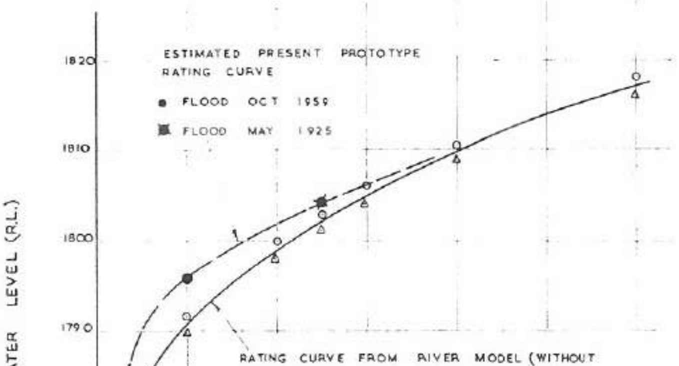


Fig. 7.—Tailwater Rating Curves.

Also, the actual tailwater depth usually lags behind the tailwater curve for rising flow and leads the curve for falling flow. Therefore, any extra tailwater depth from the tree resistance would allow greater discharge increments from the gates. This could be important for fast rising or " flash " floods.

3.—Preliminary Design Considerations.

3.1 Lake Level Control during Floods :

Normal lake level is R.L.1825. Because of the presence of formal parks, lakeside roads and public buildings around the lake foreshores, in the Central Basin area, it was considered desirable to maintain this level more or less undisturbed for as long as possible during floods.

This was done by reducing the discharge level at the dam below R.L.1825 to allow for the unavoidable rise in water level through the lake system due to friction, bend losses, etc.

Data on the water level rises for various floods and discharge levels were obtained from the river model as shown in Fig. 5. The discharge level on the river model was first varied according to the gate manufacturer's discharge rating curve for all five gates fully open (curve *a b c* in Fig. 5). The resulting Central Basin Lake level exceeded R.L.1825 at 80,000 cusecs (curve *a e f*) which was therefore the maximum fully controllable flood. Floods greater than this have an estimated return frequency greater than 35 years.

At 50,000 cusecs, a discharge level of R.L.1824 gave R.L.1825 in the Central Basin and therefore curve *d b c* (Fig. 5) was the required discharge level at the dam giving a constant lake level of R.L.1825 up to 80,000 cusecs.

3.2 Gate Operation :

The reduction in the allowable discharge levels at the dam reduces the discharge capacity of each spillway bay, thus requiring more gates to be open to pass a given flood. For example, almost 80,000 cusecs could be passed by only three gates with a discharge level of R.L.1825 instead of the five gates required with a discharge level of R.L.1821.

This had the advantage of spreading the flow which helped the energy dissipation in the lower bays 2, 3 and 4, but hindered the dissipation in the higher bays 1 and 5 by requiring them to be used earlier at lower total flows and therefore lower levels.

The discharge level variation and the gate discharge rating together determined the necessary gate operation as summarised in Table I.

TABLE I.

Total discharge (cusecs)	Gates used	Discharge level (R.L.)	Central Basin Lake level (R.L.)
0 to 45,000	3 and 4	1825 to 1824	1825
45,000 to 65,000	3 and 4 fully open plus 2	1824 to 1823	1825
65,000 to 80,000	2, 3, 4 fully open plus 1 and 5	1823 to 1821	1825
80,000 to 200,000	All 5	1821 to 1831	1825 to 1837

3.3 Choice of Dissipator Type :

Three main types of dissipator were considered : a bucket hydraulic jump and an intersecting jet type dissipator developed recently in India (Ref. 5).

Model tests showed that it was impractical to deflect flow from the outer higher bays into the lower central bays because the high discharge per foot width and relatively slow flow velocities required too high a deflector wall.

In the intersecting jet type, a series of semi-circular troughs is placed at the toe of the dam and the flow down the face of the dam is formed into a series of parallel paired jets by splitters and guide

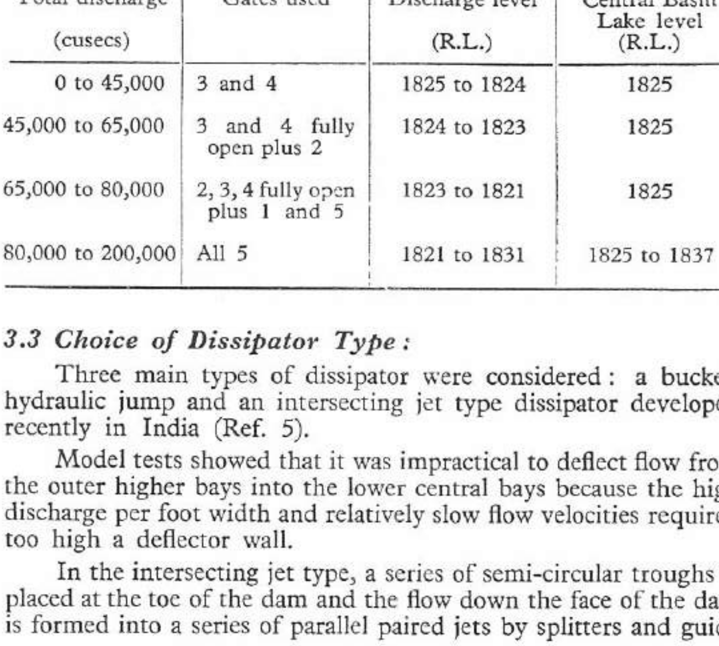


Fig. 8.

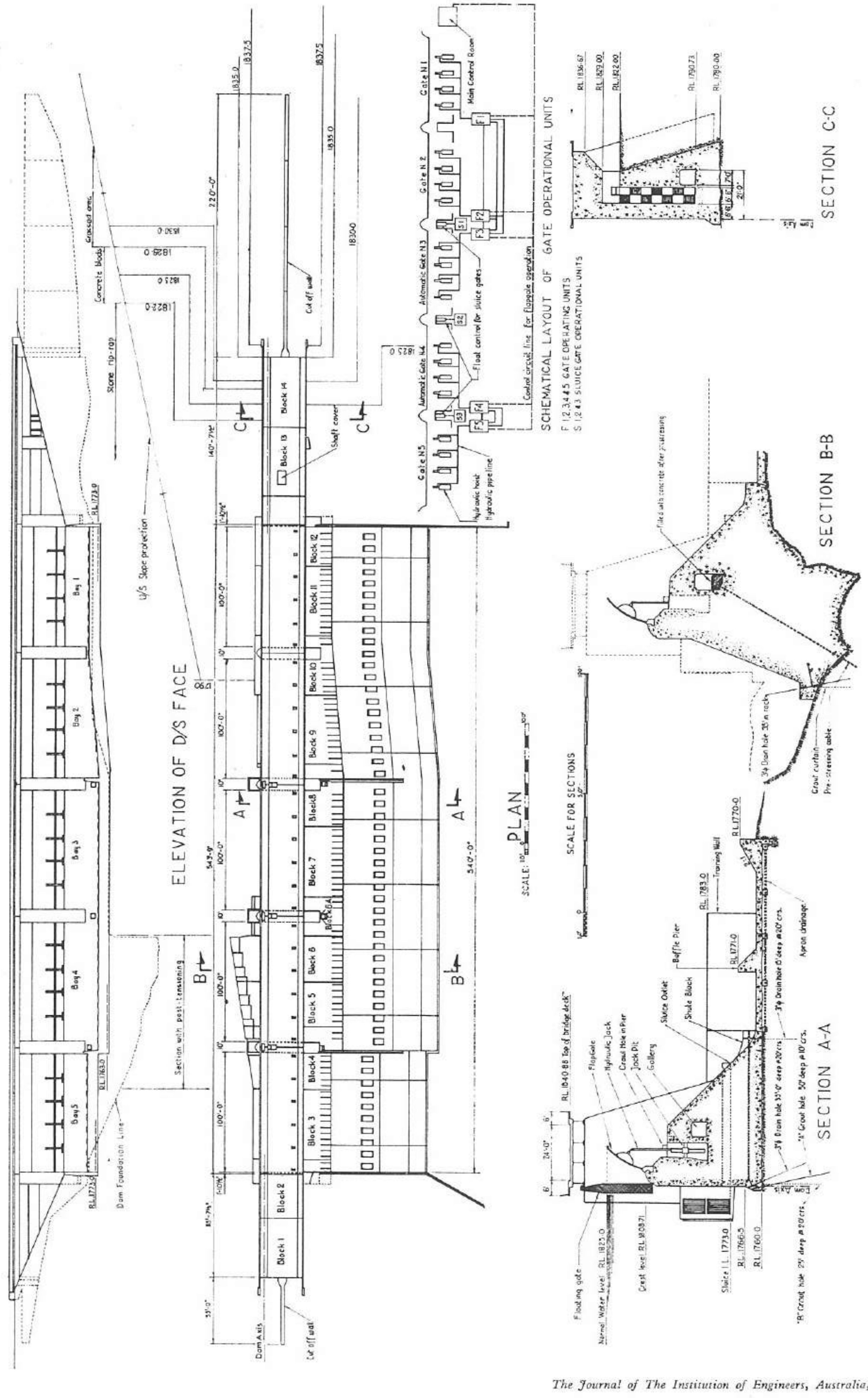


Fig. 9.—Canberra Lake Dam.

walls. Each jet of a pair enters one end of a trough which turns them laterally so that they intersect head on in the trough, causing a vertical boiling action over the top. The turbulence in the boil provides the energy dissipation. However, model tests showed that the troughs had to be very wide and deep (because of the low velocities and high discharge per foot width) which made them too expensive. Also, the splitter walls could be damaged by logs.

A bucket dissipator was discarded because the available data indicated that it required more tailwater depth than a hydraulic jump (Ref. 6).

Preliminary tests of a jump dissipator gave acceptable scour profiles and smooth water surfaces for most of the bays, so, in view of the timetable for calling tenders, this type was chosen.

3.4 Preliminary Design:

Reference was made to the extensive work on hydraulic jump basins by the U.S.B.R. (Ref. 6). This reference generalized the design of three distinct types of jump basins for horizontal aprons by expressing their dimensions in non-dimensional form and as a function of the Froude number of the supercritical flow entering the basin. Each basin was suited to a particular range of Froude numbers, discharge per foot width and inlet velocity.

These quantities varied considerably on the Lake Dam due the different bay levels and gate operation. In order to assess the critical flows, the variation in Froude number (F) and the ratio of the actual tailwater depth to the conjugate depth (R) was estimated for each bay. The minimum tailwater levels from Fig. 7 and the gate operation in Table I were used. The results were similar to that plotted in Fig. 8 for the final design.

For the two lowest bays 3 and 4, there were two distinct flow regimes, 0 to 50,000 cusecs and 50,000 to 200,000 cusecs. In the former, the Froude numbers ranged from 6 to 4 (ignoring small flows) and the tailwater depth was ideal at about 1.1 times conjugate depth (D_2). In the latter regime, the Froude numbers ranged from 4 to 2 but the tailwater depths at $1.7D_2$ were excessive and drowned out the jump.

The regime 0 to 50,000 cusecs was suited to the economical U.S.B.R. type III basin design which used baffle piers, chute blocks, solid end sill and a basin length of about 60 ft. The low Froude numbers for the high flows up to 200,000 cusecs, however, suited the U.S.B.R. type IV basin with larger chute blocks, no baffle piers, solid end sill, and a basin length of about 120 ft.

Preliminary model tests at 200,000 cusecs with this latter design showed virtually no scouring tendency and a very smooth water surface due to the jump being drowned. The basin length as therefore shortened to 80 ft. and baffle piers and smaller chute blocks installed to suit the flows from 0 to 50,000 cusecs. This arrangement was satisfactory over all flows so was adopted for the preliminary design.

The flow in bays 1 and 5 at the original level of R.L.1783 had Froude numbers ranging from about 4 to 2.5 (ignoring flows less than 80,000 cusecs from partly open gates). The tailwater depths were deficient at 80,000 cusecs ($0.8D_2$) but just adequate at 200,000 cusecs ($1.0D_2$). The conditions again suited a U.S.B.R. type IV basin about 120 ft. long. Model tests showed reasonable scour profiles at 200,000 cusecs but at 80,000 cusecs there was severe scouring and the jump was partially swept out of the basin due to inadequate tailwater depth. Only side inflow from the drowned jumps in the adjacent bays 2 and 4 prevented the jump being completely swept out. Baffle piers were then tried at 80,000 cusecs as compensation for the lack of tailwater and markedly reduced the scouring tendency. The basin length was then reduced to 80 ft. to match the central bays, with an acceptable increase in scouring. The water surface was rough due to the low tailwater which allowed unsubmerged supercritical flow to hit the baffle piers.

As the return frequency of 80,000 cusec flows was more than 40 years, this condition was accepted for the preliminary design used for calling tenders.

The preliminary design differed from that shown in Fig. 9 only by having bays 1 and 5 at R.L.1783, no training walls and the baffle piers 30 ft. from the chute blocks instead of 25 ft.

4.—Final Design Studies.

4.1 Bays 3 and 4:

Using the gate operation in Table I, the nominal maximum flow for these two bays discharging *alone*, was about 45,000 cusecs, discharging from a lake level of about R.L.1824.5. However, for final testing, a flow of 50,000 cusecs from R.L.1825 was used, as this was considered the likely maximum in actual practice.

Because of the jump profile at this flow, the water level upstream of the baffle piers in these bays was lower than the tailwater level in the adjacent unused bays. This caused a side inflow which tended to swamp the jump and make it less efficient. Cutting off part of this flow by training walls at the sides of the bay greatly improved the scour pattern and flow pattern as indicated in Fig. 10. Also, as the baffle piers, etc., were considered liable to flood damage, tests were run with these components removed. The training walls made a marked improvement to the scour profiles as indicated also in Fig. 10.

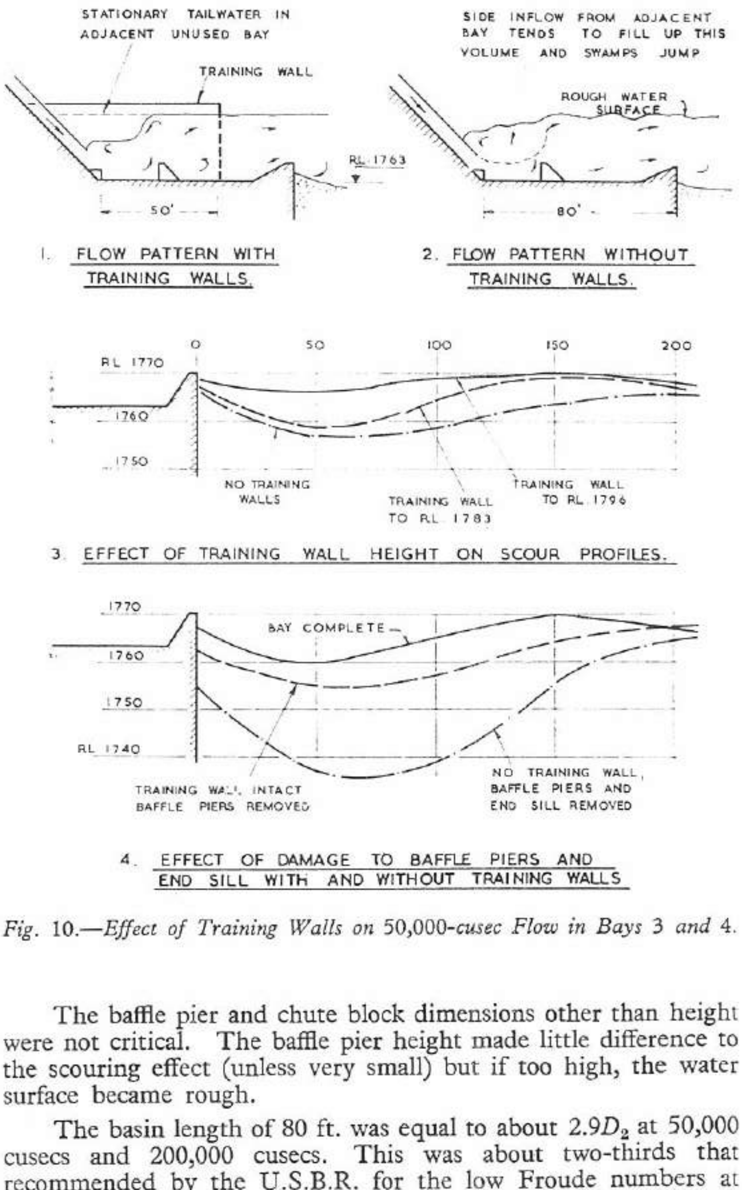


Fig. 10.—Effect of Training Walls on 50,000-cusec Flow in Bays 3 and 4.

The baffle pier and chute block dimensions other than height were not critical. The baffle pier height made little difference to the scouring effect (unless very small) but if too high, the water surface became rough.

The basin length of 80 ft. was equal to about $2.9D_2$ at 50,000 cusecs and 200,000 cusecs. This was about two-thirds that recommended by the U.S.B.R. for the low Froude numbers at 200,000 cusecs (Ref. 6). The baffle piers, end sill and excessive tailwater therefore compensated for the shorter basin length. Some further reduction in length could probably have been made with increased but still acceptable scouring, but the 80-ft. length was conservatively retained to match that found necessary for the remaining bays.

Final scour profiles for a range of flows are given in Fig. 11.

4.2 Bays 1 and 5:

The preliminary tests for these bays at R.L.1783 (assuming the gate operation sequence in Table I) were considered poor but probably acceptable, considering the low frequency of the large floods requiring their use. However, if, due to faults in other gates, bays 1 and 5 had to be used out of sequence and therefore at lower total flows and tailwater levels, the scouring tendency would be more severe.

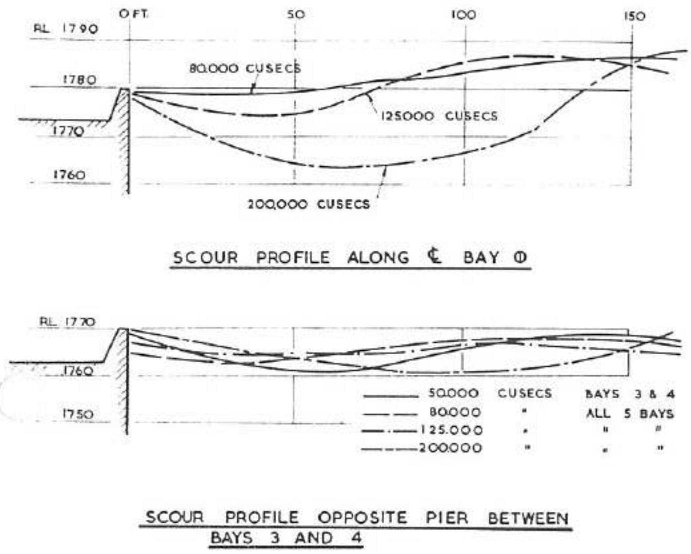


Fig. 11.—Scour Profiles for Final Design.

Also, it was considered desirable to be able to release test flows from these gates both for contract acceptance tests and possibly (considering their infrequent use for floods), maintenance tests. With these bays at R.L.1783, such flows would have almost zero tailwater depth and would therefore cause very severe scouring. Testing would therefore be limited to raising and lowering the gates with the lake storage blocked off by the gate maintenance caisson.

If the bays were lowered to provide tailwater for the test flows, this would also provide better flood flow dissipation and allow more flexibility in the order of gate opening.

Scour tests were therefore carried out to find the minimum tailwater levels for satisfactory dissipation of test flows discharged from normal lake level of R.L.1825. The relevant results are given in Fig. 12. Two bay levels, R.L.1783 and R.L.1773, and two test flows, 25,000 cusecs (fully open gate) and 15,000 cusecs (gate two-thirds open giving maximum load on hydraulic rams), were used. Probably only test flows approaching the latter would ever be used in order to avoid too much flooding downstream.

With the bays at R.L.1773 and using the estimated present tailwater rating curve (Fig. 7), the tailwater depth was about 0.6 conjugate depth for 15,000 cusecs. At this depth, the model indicated maximum scour depth at the end sill slightly below the bottom of the cut off wall. However, such tests would be very infrequent and of short duration. Also, because the end sill was founded in rock, the model scour depths were considered quantitatively, to represent more accurately the long term effect of test flows. Therefore a bay level of R.L.1773 was considered just acceptable, but it was realised that in order to avoid lower tailwater depths due to a lag in tailwater build up, tests on these bays would probably have to be preceded by a similar test on one of the lower central bays.

With the bays at R.L.1783, however, a test flow of 15,000 cusecs would have zero tailwater and the model test results were considered unacceptable.

Therefore, in view of the other additional advantages from lowering the bay (better flood flow dissipation, more flexibility in gate opening sequence), bays 1 and 5 were lowered to R.L.1773 for the final design.

Further tests to find optimum baffle pier and chute block dimensions showed that these were not critical and that the dimensions used for bays 3 and 4 were satisfactory. The baffle piers had little effect on normal flood flow dissipation because of the low Froude numbers and excess tailwater (Fig. 8). Their main use was at tailwaters less than conjugate depth when the jump would be washed out of the basin without them. This would occur during the gate testing or if the gates were used out of normal sequence (Table I).

The basin length of 80 ft. appeared more than adequate for normal flood flow dissipation, but was retained as shorter lengths made test flow dissipation and out of sequence gate operation more critical.

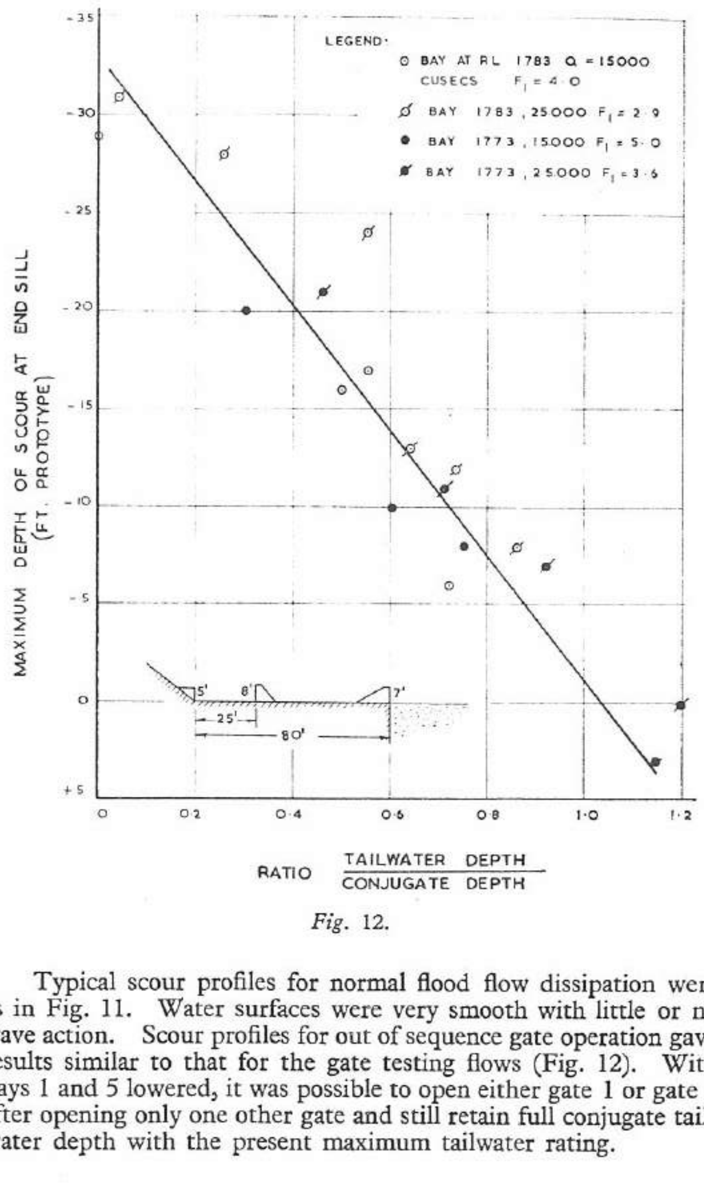


Fig. 12.

Typical scour profiles for normal flood flow dissipation were as in Fig. 11. Water surfaces were very smooth with little or no wave action. Scour profiles for out of sequence gate operation gave results similar to that for the gate testing flows (Fig. 12). With bays 1 and 5 lowered, it was possible to open either gate 1 or gate 5 after opening only one other gate and still retain full conjugate tailwater depth with the present maximum tailwater rating.

5.—Conclusions.

The maximum design flood of 200,000 cusecs had little direct influence on the stilling basin design. The main factors were the lake level control requirements during floods, the different bay levels and the consequent gate operation sequence, which made the low and intermediate flows more critical.

Because of the wide range of flow conditions in each stilling basin bay, the available data on hydraulic jump basin design were not directly applicable with economy, and the model study was most useful in assessing the effects of those flow conditions outside the limits set for standard design.

PART III

Design and Operation of the Canberra Lake Dam

BY A. FOKKEMA

1.—Introduction.

The dam constructed to form an ornamental lake in Canberra has a total construction length of 1,325 ft. along the dam axis.

The dam consists of a concrete gravity section with five 100-ft. x 16.8-ft. flap gates between two concrete gravity buttress non-overflow sections of 230 ft. total length, and of earth embankments 555 ft. long of which a total length of 275 ft. has a centre concrete cut-off wall founded on rock and 3 ft. thick. Maximum structural height of the dam is 111 ft.

2.—Geology.

The foundation rock of the dam and the spillway apron consists of a quartz porphyry. The rock was for the greater part of the river valley floor covered by alluvium of varying thickness.

At the left bank this alluvium comprised an upper layer of maximum 10 ft. thickness of an aeolian material of very fine sand and silt with varying proportions of clay. This material can support a considerable load when dry but rapidly breaks down when immersed in water. A protective pavement was therefore constructed on the left bank of the lake to 375 ft. upstream from the dam axis, to prevent slumping of the lake bank into the old river bed immediately upstream of the dam.

It was found during the exploratory diamond core drilling that a broad zone of jointing occurred in the foundation rock under the river bed near the western bank, running parallel to the river channel with a steep dip. This now is the location of Block 6 and the adjoining half of Block 5.

It was thought that the total width of this jointing zone was approximately 60 ft. with a zone 20 ft. wide containing three bands of up to 3 ft. width of crushed rock and clay. It was pointed out by the Bureau of Mineral Resources, carrying out the geological investigations, that the exact position of these bands would only be revealed when the overburden was stripped from the area.

Two other steeply dipping narrow fracture zones parallel to the river consisting of closely jointed rock with clay and limonite staining were expected to occur under the eastern part of the dam.

When the overburden and the weathered rock were removed in this location, these zones were found to conform closely to the expectation; but during the cleaning off works at Block 3 a relatively small joint found in the rock of Block 2 widened out substantially and showed increasing amounts of clay and badly fractured rock. All rock in Block 3 above the footwall of this fault was excavated and diamond core drilling commenced to assess the properties of this feature and its connection with the jointed area at Blocks 5 and 6. These investigations were carried out by the Bureau of Mineral Resources and advice was obtained from the Snowy Mountains Hydro-Electric Authority.

It was found that this feature, referred to as fault " B ", striking at 45° to the dam axis, dipped downstream at 35°. Another major fault " A " striking perpendicularly to the dam axis dipped 65° to the east and joins or intersects fault " B " under Blocks 6-6A.

In the acute angle formed by these two faults, in the hanging wall of fault " B " other features appeared to have contributed to the shattering and shearing of the rock at the dam location (see Fig. 13).

Fault " C " apparently was confined to the zone between faults " A " and " B " and faults " D " and " E ", a fracture zone, formed by two joining or intersecting faults. Fault " D " appears in the foot wall of fault " B ".

The rock under the spillway apron, downstream of the affected dam blocks, was also investigated. No further major features were found here but minor jointing continued with some further shatter zones containing some clay.

Faults " A " and " B " were found to continue under the apron as expected.

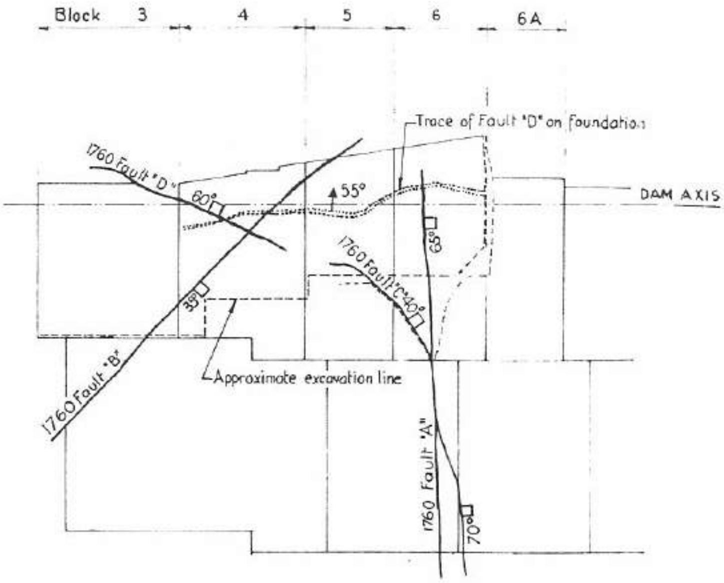


Fig. 13.—Major Geological Features in Foundation.

3.—Stability Consideration.

For the stability design of the concrete gravity sections, the following uplift assumptions were used.—

(*a*) for normal lake water level RL.1825: no tension at foundation level for full uplift, that is with drains inoperative and the uplift pressures varying linearly from reservoir pressure at the upstream face to tailwater pressure at the downstream face;

(*b*) for the maximum design flood of 200,000 cusecs, reservoir level RL.1831, tail water level RL.1810: no tension at foundation level for an assumed uplift condition from reservoir pressure at the upstream face to tailwater pressure at the drain holes and from there a constant tailwater pressure to the downstream face;

(*c*) for the maximum probable flood of 300,000 cusecs, reservoir level RL.1837.5, tailwater level RL.1817: maximum tension at the upstream foundation face of 15 lb./sq. in. for an uplift condition as for (*b*).

4.—Grouting and Drainage.

Water pressure testing of the drill holes showed only small water losses, even in the fractured fault zones.

Blanket grouting of the foundation was confined to the upstream part of the dam foundation and consists of grout holes on 20-ft. centres in two lines 20 ft. apart and depths of 25 ft. One grout line is located near the upstream face of the dam.

A grout toe 2 ft. wide was provided for the drilling of the curtain grout holes, dipping at 12° under the dam. These grout holes are spaced 10 ft. apart and are 50 ft. deep.

Drainage of the foundation rock under the dam is obtained by 3 in. dia. diamond drill holes, 35 ft. deep at 20-ft. centres, drilled through the grout toe, dipping 23° downstream. These drain holes discharge downstream through the chute blocks, via a system of pipes set in the dam concrete. A second line of drain holes 15 ft. deep and 20-ft. centres was drilled through the chute blocks to reduce the uplift under the spillway apron. The apron is designed as a 3 ft. thick slab with a system of no-fines concrete underdrains with 6 in. dia. open jointed concrete collector pipes. The apron is anchored to the rock by use of $1\frac{1}{8}$ in. dia. anchor bars spaced 7-ft. centres both ways and grouted for a length of minimum 5 ft. into the rock.

5.—Dam Section at Fault Area.

When the nature of the rock in this area became known, it was realised that some re-design of the section would be required.

Preliminary excavation limits were established for the rock above fault " B " with a downstream boundary from 40 ft. downstream of the dam axis in Block 4 to 30 ft. in Blocks 5 and 6.

This line followed approximately the border between fractured and sound rock in the hanging wall of fault " B " and provided a minimum thickness of 20 ft. of sound rock above the fault plane.

A number of possible solutions were studied. Some of these are :

(*a*) To rely on the downstream rock to take its part of the horizontal dam reaction forces. Though subsequently no further major clay-containing features were discovered in the rock under the spillway apron, no positive proof was obtained that no such structures existed downstream of the apron.

Some broken and weathered zones with limonite and clayey materials on the joints were found in the rock under the apron and it was felt that no complete reliance could be placed on this rock.

(*b*) The use of a vertical arch over the affected rock with abutments adjoining Blocks 3 and 7. Study of such an arch showed that massive excavations would still be required in sound rock, for the abutments, while shattered rock remained under the dam.

The strike and dip of fault " B " make it difficult to obtain satisfactory abutments, on which the stability of 220 ft. of dam would depend. A secondary water retaining structure upstream of or under the arch would substantially increase the total cost. Also, concern was felt about the arch behaviour under the load of the rigid dam blocks on top.

(*c*) The use of post-tensioned high tensile steel cables to tie the blocks back to the footwall of fault " B " and so relieve the load on the rock above the fault downstream of the dam.

The last solution was finally adopted. The nominal 200-ton cables were located in the dam, so that drilling for the cable holes could be carried out from the downstream face of the dam and the anchor heads could be formed and stressed in the gallery. The cables dip upstream at 30° with the vertical. The foundation reactions of this dam section are indeterminate and do not lend themselves to a rigid analysis. For the determination of the number of cables (27 in number for 129.5 lin. ft. of dam) it was assumed that if the foundation rock above fault " B " would be able to carry vertical load only, the resultant of all other forces including the water load could be resisted on the foundation rock below the fault.

6.—Flap Gates.

Tenders were called for the design, manufacture and erection of the five 100-ft. crest gates, requiring floods to be passed over the dam with a minimum of obstruction to the passage of debris.

It was stressed that the neatness of appearance of the gates would be considered of great importance. The gates now erected are fish belly flap gates designed by Rheinstahl Union Brückenbau, West Germany. The water load is carried by the skin plate-fish y section to six cross beams. Each of these cross beams is supported by a hinge, anchored to the concrete dam crest and the four centre beams are also supported approximately at their half points by hydraulic jacks.

Also a floating maintenance gate was included in the requirements. This floating gate can be placed in front of any of the flap-gates to dewater the upstream face of the flap-gate and is then supported on the bridge over the dam and on the concrete crest of the dam.

As the gates will be required to discharge floods only for five days in an average year, it was felt that continuous operation of the hydraulic gate equipment was not advisable and provision has been made to be able to support the gates independent of the hydraulic jacks. The tops of the outer cross beams can be supported on retractable hinges set in the pier side shields.

These pier hinges consist of a hydraulic ram, which can be pushed out and retracted by a hand-operated pump unit located in the gallery. Thus, also, major maintenance and repair of the hydraulic gate equipment can be carried out without the use of the floating gate.

A further design consideration was that a gate should be able to continue operation after the failure of any of its jacks without dangerously overstressing the gate structure and the hydraulic equipment. The maximum loadings by the gate on its hinges are as follows.—

Outer Hinges —Hor. :	+ 44.7 tons to −25.4 tons	
Vert. :	+ 67 tons to − 6.0 tons	
Centre Hinges—Hor. :	+106.9 tons to −13.8 tons	
Vert. :	+ 54.7 tons to −80.1 tons	

Hor. : Positive for a downstream direction.

Vert. : Positive for a downward direction.

The maximum load on the hydraulic jacks : 165.0 tons

The maximum load on the pier hinges : 77.6 tons

The stresses in the dam concrete by the hinge loadings were assessed using photo elastic studies, carried out by the Snowy Mountains Hydro-Electric Authority.

The reinforcement requirements, particularly near the hinges and around the gallery and the jack pits, were thus determined. It was also found that these hinge reactions could produce local tensile stresses near the upstream face of the dam, which at the highest foundation level still could be of the order of 25 lb./sq. in. An upstream face reinforcement has been provided in the overflow sections of the dam. In blocks with high foundation, this reinforcement was continued into the rock, using $1\frac{1}{8}$ in. dia. hard grade steel deformed anchor bars at a minimum spacing of 3 ft. 6 in. with grouted anchor length in the rock of 14 ft. and 7 ft. alternatively.

For anchorage of the hinges into dam concrete, hard grade steel reinforcing bars were used.

The connection between the hinges and the anchor bars was made by threading the ends of the bars and tightening the nuts on a mild steel beam assembly connected to the hinge.

After the hinges had been accurately adjusted for alignment, the connecting assembly was concreted in.

It was found that the hard grade steel bars had been welded to distribution bars. Accuracy for the placing of these bars was required to assure that a satisfactory connection to the anchor beam assembly could be made. Investigations were carried out on the possible effect of this welding to the strength of the hard grade steel bars. It was decided to tie down the fifteen affected hinges by using two nominal 45 tons prestressing cables per hinge. These 25 ft. long cables consist of 12 No. 0.276 in. dia. high tensile steel wires, using a grouted blind anchorage 12 ft. long and a steel anchor head resting on a 12-in. x 15-in. x 2-in. MS thrust plate.

7.—Control of Water Level.

The main criterion in the control of Lake Burley Griffin is to keep the water level as near to R.L.1825 as is possible with the gates provided in the dam.

At the time of writing, the proposals for the control of the water level have not yet been finalised and some changes may need to be made to the following.—

Three 4-ft. x 4-ft. sluice gates have been installed and can automatically adjust the outflow from the Lake for a range of 6-in. change in water level. These sluice gates were designed by the S.M.H.E.A.

By setting the float-operated control equipment for the first sluice gate at R.L.1824.9 the mean annual inflow of 180 cusecs will raise the water level to R.L.1825.0. Under steady flow conditions the sluice gates can pass a discharge of approximately 2,000 cusecs without allowing the water to rise above R.L.1825.4. During periods of minor floods the filling of the Lake storage above R.L.1825 is expected to enable the sluice gates to handle floods with peak discharges of less than 3,250 cusecs. This figure was arrived at from a comparison of the shapes of flood hydrographs and the average flood routed through the storage.

From the recorded flow duration curve for the Molonglo River near the dam, the inflow into the Lake is expected to exceed 3,250 cusecs 1.3 per cent of the time or 5 days per year. During these periods one or more of the floodgates will be required to discharge the excess or the total flow.

In some years it will not be necessary to use the flap-gates for flood discharge at all. The longest recorded period that flap-gate operation would not have been required was from December, 1925 to March, 1929. For the operation of the sluice gates the measure-

ments of the inflow is not of great significance as the outflow from the lake is directly related to the water surface near the dam. However, for efficient flap-gate operation it is important that the inflow to the lake is known during periods of high flows.

The top of the shore protection walls in East Basin is R.L.1826.5, and a rise of the lake here to R.L.1830 will flood an area containing 20 houses.

Two existing gauging stations are available on the main streams and can provide accurate discharge measurements.

Burbong on the Molonglo River has a catchment of 195 sq. m. and Googong on the Queanbeyan River has a catchment of 337 sq. m. One other station on the Jerrabomberra Creek is available, but the recorded stream flow data available are limited in quantity and poor in quality. The catchment for this station is 55 sq. m. These three stations cover approximately 77 per cent of the total lake catchment. The river flow at the stations can be obtained in Canberra via normal telephone channels connected to the telemetered recording instruments installed. Each station is also provided with an automatic alarm when a certain flow at the station is reached. This alarm will be received at a central point in Canberra, from where the personnel in charge of the Lake level control are to be alerted.

A study of flood hydrographs indicates that the initial settings of these alarms should be as follows.—

For Burbong	900 cusecs
For Googong	1,800 cusecs
For Jerrabomberra	500 cusecs

False alarms will be possible with these settings, but are expected to be relatively few in number and will cause little inconvenience. The settings can be adjusted when more reliable information becomes available, especially for the Jerrabomberra Creek. This creek has on a number of occasions experienced floods from localised storms, with an increase in flow of up to 4,000 cusecs per hour.

With the three alarms covering all but 150 sq. m. of lower lying catchment area, the possibility of a damaging flood occurring from which no warning had been received is small. However, the possibility that an instrument will malfunction is real and a number of check alarms will be provided.

Also an alarm will be placed just upstream of the dam providing a check on all installations located in the catchment. This alarm is to be set at R.L.1825.6 or for a water level when closed spillway gates are overtopped by approximately 1 in.

A gauging station downstream of the dam will be able to supply information at any time about the actual discharge of the sluice and flap-gates. The average time required for a flood wave passing Burbong and Googong stations to reach the dam is estimated to be around six hours. The travel time from Jerrabomberra stations to the dam is around one hour. The time of travel of a flood wave through the lake itself has been estimated as varying between 30 minutes to one hour.

Thus, after an alarm from Burbong or/and Googong is received the dam-operating personnel can obtain an indication of the actual inflow to the lake to be expected in some five hours' time and the contribution of the Jerrabomberra Creek can be added to suit.

The operator then can control the flood through the lake with a minimum of damage and inconvenience to all concerned.

It is intended, after an alarm is received, to contact the telemetering gauging installations and to assume in first instance that the computed total discharge of these stations multiplied by a predetermined area ratio is the actual inflow to the lake at that time.

From further information obtained a hydrograph of the expected actual inflow to the lake can be constructed, to adjust the initial settings of the gates as required.

Flap gates 3 and 4 then, for the immediate future, will possibly only be set on automatic operation when heavy rainfall is expected and perhaps for a period after a flood has occurred. The settings for the float control of these flap-gates at the dam will be initially R.L.1825.3, with a range of four inches.

After more information becomes available by actual measurements in the lake and the catchment, specifically of the flood wave movements in the river below the gauging stations and through the lake, a review can be made to decide if a further use of automatic controls of the lake by flap-gates is advisable.

Acknowledgment.

The presentation of these papers has been made possible by kind permission of Mr. R. B. Lewis, O.B.E., B.Sc., B.C.E., B.M.E., M.I.E.Aust., Commonwealth Director-General of Works.

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Discussion

Mr. V. Michels (Member, Melbourne Division).— The authors are to be congratulated on their concise presentation of many technical aspects of the Canberra Lake Scheme. We should also be thankful for the excellent timing of the Conference —we are able to enjoy Canberra at her best and to admire the albeit slowly-filling Lake whilst simultaneously digesting the papers on the engineering works involved.

In his winning entry, Walter Burley Griffin saw the site as an " irregular amphitheatre . . . with the water-way and basin the arena ". Now, a half-century later, the construction of the dam and associated works realises his conception—an ornamental lake decorating the centre of the city from every vantage point. Indeed, one is moved to describe today's impressions in more poetic language.—

> *Surrounded by admirers—rugged, tall, unruly,*
> *She curtseys low—a neat young lady, gaily gowned;*
> *The mirror at her dainty feet, reflecting truly*
> *Her regal grace, her mantle's autumn tints renowned.*

The consideration of aesthetics as part of an engineering design or, as in this instance, also the *principal reason* for the project—is worth stressing. Thus we note that the specifications for the control gates provided for neatness of appearance. Also, the various Lake structures were functionally proportioned, with simple pleasing lines. It is well to note that, frequently, it costs no more to achieve beauty—as well as strength, stability and safety. The engineer's feeling of achievement is justifiably greater where the completed project enhances the natural features, as is the case here. When the structure is in the public eye, functionally and visually, the creation of " ugly heirlooms " should particularly be avoided.

Turning to the technical aspects, one of the most striking impressions was the great value derived from the hydraulic studies on the river model. It would be interesting to see the cost of this investigation compared with the direct and indirect savings effected. Naturally, as is often the case, some of the information and advantages would be hard to evaluate in £.s.d.—or should I say, dollars and cents !

As regards the derivation of flood hydrographs, it is understandable that a fairly conservative approach should be made where damage by flooding would cause high actual monetary losses. Thus, the Meyer coefficient for the adopted design flood peak of 200,000 cusecs is 72; for the maximum probable flood of 300,000 cusecs, the figure is 108—which, for most Victorian streams, would be regarded as unusually high. The basis used for determining these flood hydrographs has not been stated; e.g., was the latter limited to the largest conveyance for the flood plain, i.e., without further spread of water, or derived from the maximum likely precipitation on the catchment, or determined by some other method? The model verification of the 1925 flood levels and the consequent evaluation of the probable peak discharge, were most interesting. Incidentally, it appears a novel method of finding errors in levelbook reductions has been discovered! The tremendous value of recording flood levels or of even merely photographing flood waters, whenever and wherever possible, has also been amply demonstrated.

The storage capacity of the Lake is not mentioned but, presumably, it has insignificant pondage effect on the design—and maximum flood peaks, as the text rarely distinguishes between inflow and outflow hydrographs.

The question of siltation of the Lake was briefly mentioned. The likely pattern of deposition could perhaps be studied in the river model. The silt storage volume at the dam, i.e., below the invert level of the sluice outlets, appears to be very small. However, the narrow and long waterway upstream of the dam may assist passage of bed load by density currents (Ref. D1). Was any consideration given to the prevention of inlet blockage, by the provision of some form of silt-arresting structure (Ref. D2)?

Some clarification is requested regarding a statement on the tests for siting the Kings Avenue Bridge. Are the "modifications to the approach embankments", mentioned in Part I, in fact "the excavation of a channel . . . 300 ft. wide and 20 ft. deep beneath the bridges", stated by Birkett and Fernie to have improved peak flood flow characteristics through the promontories (Ref. D3)?

The writer would also appreciate confirmation of the factors which fixed the normal Lake level at R.L. 1825 ft. Presumably it was the highest practicable level, having regard to existing road levels and the 24-ft. clearance evidently required for boats on the Lake. From another source one learned that the depth of still water must exceed a certain figure if mosquito breeding is to be inhibited. Similarly, the design flood level on the central basin (R.L. 1837 ft.) was evidently fixed by levels of established improvements or permanent facilities along the shore.

Perhaps the authors could comment briefly on the siting of the dam and on the alternative types and arrangements considered before adopting the ultimate arrangement. From Fig. 1, it appears the chosen site, being the first narrow section below the wider basin areas, would entail the least dam volume for a given lake level. However, it would be informative to know whether earthen and rockfill types associated with a separate spillway through an abutment or a convenient saddle in a spur—and utilizing the excavated material for embankment construction—were feasible. If so, were such alternatives more costly or were they discarded for other reasons? Also, would a secondary or emergency spillway have been practicable for the chosen site, thus reducing the required capacity of the primary overfall structure? For the Eppalock Dam (Ref. D4) this arrangement effected considerable overall economy.

The writer also wonders if, in the adopted arrangement, it would have been advantageous for hydraulic reasons to have used slightly deeper gates in the *three* (not two) middle bays and shallower gates in two outer bays. Lowering the central apron would have permitted slight raising of the outer sections. The discharge would have been symmetrical and the side bays would have come into operation at higher tailwater stages. However, this layout may not have fitted equally well the profile of the sound rock in the river. The disadvantages of having two sizes of gates and operating gear, variations in crest and pier designs, etc., may have been at least partially compensated by simpler construction of the symmetrical apron, etc.

For the stability design of the concrete gravity sections, some would consider uplift assumption (*a*)—for normal, lake-full conditions—definitely conservative in that drains are considered completely inoperative. The provision of a gallery located at the lowest practicable level (as may well have been feasible in this case) permits visual inspection of the flow from (or pressure in) each drain hole, and facilitates future drilling of additional holes (or re-drilling holes) for drainage or grouting purposes. Be that as it may, the writer considers the adopted 20-ft. spacing of drain holes rather wide. As water-pressure testing of drill holes disclosed low losses, a closer interval—with the above-mentioned provision for subsequent drilling, etc., if required—would have made blockage less likely. Whether any economy would have been effected by thus relaxing uplift assumption (*a*) would depend on its severity in comparison to assumptions (*b*) and (*c*).

The duration of the design flood (200,000 cusecs) and of the maximum flood (300,000 cusecs) have not been stated. However, even though these periods are no doubt relatively short, the adopted uplift assumptions (*b*) and (*c*) could hardly be relaxed for the drainage provisions made.

A rough estimate of the principal compressive stress at the toe of the dam, suggests a figure of about 15 tons per sq. ft.; excepting Block 6 which may have imposed a stress of approximately 25 tons per sq. ft. without post-tensioning. In the absence of sections showing the positions of various fault zones, etc., one has difficulty in assessing the complicated conditions in this area from Figs. 9 and 13 alone. However, the post-tensioning solution adopted for Blocks 4, 5 and 6 certainly appears to be the soundest of the alternatives. As the estimated likely pressures are not high for the competent rock, refined analytical methods or structural model tests were probably not warranted—particularly as the true extent of clayey materials in the rock under the apron was not determined.

The writer would appreciate comments on the factors leading to the mass concrete section for the portions adjoining the spillway, namely, Blocks 1, 2, 13 and 14. Was it found less economical to have continued the earthen section through to the spillway, because heavier training walls (also acting as retaining walls) would have been required?

Another point of interest is the vertical rock-to-concrete face at the southern end of Block 6. Perhaps it would have been practicable to batter this face, particularly as it adjoins Fault "A", which dips under it. In the writer's experience such rapid changes in foundation level have been effected by sloping faces rather than sharp steps, to avoid stress concentrations and to counteract (by gravity) the "parting-away" effect of concrete shrinkage. Were special precautions taken at this point, in regard to sealing and cutoff provisions. The question also arises as to whether variations in rock properties are likely to cause settlement of Blocks 4, 5 and 6 relative to adjoining monoliths, particularly as they are deeper and are post-tensioned in the upstream/downward direction. Also, construction photographs showed that block joints were keyed. This would appear unnecessary and—for the inter-face between Blocks 6 and 6A—an unwise provision as it is directly above the cited vertical rock-to-concrete face. The writer would also appreciate brief details of the water-stops used.

In retrospect, do the authors now consider it would have been advantageous to have moved the dam site up or down the river, to avoid at least some of the special foundation treatment necessitated by the faulted zones as disclosed by exploratory drilling?

Some amplification of the grouting results would be appreciated: what was the order of the "takes" obtained and pressures used for blanket and curtain grouting? Were special patterns applied near fault zones and at the vertical step in the foundations?

As regards the apron anchorage design, the writer feels that the anchor bars could have been spaced further apart and grouted deeper into the rock, for a more balanced result.

With respect to the hydraulic equipment, I am sure many would be glad to have additional notes on the design, operation and

storage arrangements of the floating maintenance gate. Also, details and effectiveness of the flap-gate seals—at sides and hinges—would be of great interest. Similarly, brief notes on the automatically operated sluice gates would be appreciated. In passing, the authors are asked to elaborate on a somewhat cryptic remark in the paper: it was stated that investigations were carried out on the possible effect of welding hard-grade steel anchors (for the flap-gate hinges) to the distribution bars in the concrete; the outcome of the tests was not given but evidently their strength was impaired as prestressing cables were installed.

In conclusion, I would like to ask the authors to furnish some cost figures. Most engineers are vitally interested in this question, and data would be welcomed on the overall and unit rates for the dam and its main components.

Apologies are tendered because of the number of questions raised. However, this should be accepted as a measure of the interest the paper has evoked in at least one engineer engaged on dam design.

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2. MICHELS, V. S. and ILOTT, L. P.—Reservoir Outlets and Control Works. *Jour. I.E. Aust.*, Vol. 27, No. 7-8, July-Aug., 1955, p. 181, Fig. 15 and p. 186, Ref. 37.

D3. BIRKETT, E. M. and FERNIE, G. N.—Bridges in the Canberra Central Lake Area—Design. *Op. cit.*, Vol. 36, No. 7-8, July-Aug., 1964.

D4. JOHNSTON, R. B.—Spillways for Earth and Rockfill Dams. *Indian Jour. Power and River Development, Hydraulic Structures*, Special Number, 1963, p. 332.

Dr. E. K. Carter (Official representative of the Bureau of Mineral Resources, Geology and Geophysics).—The question has been asked whether, in view of the difficulties encountered in the foundations below blocks 4 to 6 of the Scrivener Dam, the geological investigations are considered to have been adequate.

I would like to outline the site investigation. First it should be borne in mind that the site on which the dam has been built was the third to be investigated. Extensive geological mapping, drilling and seismic testing of sites near Lennox Crossing, Acton, and about 500 yards upstream of the Scrivener dam were done before the existing site was selected. The time available for the geological investigation at the present damsite was consequently very restricted by overall development planning considerations for the whole of Canberra. Unfortunately geophysical services to test the Scrivener site were not available when required, owing to other commitments.

A total of 24 cored diamond drill holes were drilled and water-pressure tested in the course of the Scrivener site investigation. From the outset of the investigation it was recognised that a fault or shear zone existed beneath the western channel of the Molonglo River, and several drill holes were designed to define the conditions prevailing at that point. Every report submitted by the site geologists indicated that the foundation conditions below the western channel were inadequately known. The difficulties that have arisen with the dam foundations, involving as they did redesign of the foundation treatment for blocks 4 to 6A, are due to the fact that the zone below the western channel represents the intersection of two faults—one parallel to the centre-line of the dam and dipping 65° W and the other striking at 45° to the axis of the dam and dipping obliquely downstream at 35°. Other supplementary faults are present. The 35° fault (which is not a simple planar structure) was not recognised because below blocks 1 to 3 it was near to, and parallel with, the land surface along the axis of the dam. Drill holes that intersected it revealed only 1 to 4 in. of broken, weathered and partly-decomposed porphyry which were thought to represent zones of near-surface weathering in porphyry (which is everywhere very highly fractured). Design excavation level was chosen to eliminate this weathered material.

Drilling had revealed that strongly weathered and crushed porphyry extended at least 140 ft. below the western channel of the Molonglo River. The tentative geological explanation of the crushing and deep weathering was that a near-vertical shear-zone about 40 ft. wide and striking roughly parallel to the centre line of the dam, was present at this point. Further investigation was recommended of this area but the time available did not allow it to be carried out. It was decided, in view of the urgent need to proceed with design and construction, to assume that some additional excavation would probably be needed below the western channel of the river, and allowance was made for treatment by extra concrete, by bridging, or similar measures. These would have been adequate had the unrecognised 35° fault not been present.

To answer the question asked, then, the geological investigation was not considered by the geologists concerned to have been adequate, but over-riding planning considerations led to the decision to proceed with design and construction of the dam. Allowance, which in the event proved inadequate, was made for the unresolved geological uncertainties.

The Authors in Reply:

To Mr. Michels.—Mr. Michels has raised a large number of questions in his searching discussions and we thank him for his interest.

Questions relating to the design flood hydrograph, the fixing of the lake level and the design flood level are not directly within the province of the authors but the following information is offered.—

The design flood was arrived at by probability studies of the records of flood flow on the Molonglo and adjoining catchments. These records were built up to cover about 100 years and were certainly influenced by major floods of 1923 and 1925. The maximum possible flood was derived from a maximised storm pattern precipitated over the catchment. These studies were carried out by the Department Head Office in Melbourne.

The lake level was initially set at R.L. 1825 by Scrivener, the land surveyor who conceived the water feature in his original survey and report. This level was investigated for economic, aesthetic and practical considerations and subsequently adopted by the Development Commission. The criterion of the design flood not to exceed R.L. 1837 in the Central Basin was based on that level which this flow would have reached had it occurred before the lake was built. It was fixed by the Commission after consideration of factors including the major buildings planned on the foreshores and the frequency of maintaining the lake at a constant 1825 level.

Costs of the model investigations amounted to £6,000 for the river model and £2,500 for the spillway model. These sums covered the fabrication of models and the supply of all equipment. It excludes the salaries of the testing officers and costs relating to the possession of the building.

With regard to the pondage effect on the hydrograph the storage capacity of the lake is 27,000 ac. ft. with a surface area of 1,750 acres, and when considered with an inflow of 200,000 cusecs the effect is minute. This in fact was tested on the model.

At Kings Ave. Bridge the major hydraulic improvements to flow were achieved by resiting the bridge and thus aligning the flow at a better angle to the bridge axis. Further improvements were achieved by modifications to the approach embankments. The peak velocity through the opening was then reduced by increasing the waterway area beneath the bridge by excavating as described by Messrs. Birkett and Fernie.

The problem of siltation of the lake has been given much thought. It was concluded that no quantitative results could be obtained from a lake model study. It is intended to measure the actual deposition of silt in the lake at regular intervals and after floods.

An open rockfill causeway immediately upstream of East Basin is under consideration. This rock barrier will temporarily dam flood flows over the upstream flood plain and so cause reduction of flow velocities and turbulence, thereby reducing the sediment

carrying capacity of the flow. The expectation is that much of the suspended silt load could so be deposited on the flood plain before the water enters the lake; e.g., for a 50,000-cusecs flood the length of the settling basin will be approximately 7000 feet if such a barrier is built, and it is calculated that some 58 per cent of the silt load could be deposited on the flood plain, 17 per cent retained in the lake and 25 per cent carried through the lake over the dam.

This would mean that dredging of the lake at long intervals will still be required to retain a satisfactory lake depth. If it is found necessary, dredging near the dam can be carried out at the same time.

Such a barrier will have small influence on the clay coloid content of the inflowing water and therefore on the colour of the lake water after a flood.

The wind-blown fine sand and silt on the left bank at the dam is quite a difficult material. Tri-axial saturated tests showed very low values for the angle of shear resistance. Its use in water retaining earth embankments was decided against, and the length of the left bank non-overflow section was determined by an allowable fill slope of 1 in 4, using a heavy rock and filter protection on the under-water slopes.

The rock to concrete face at the southern end of block 6 was battered back to approximately 5 to 1. A blasting out of a less steep b[illegible] could have endangered the foundation rock below the already constructed block 7. No post-tensioning in block 6A was used as the stressing cable would have to pass through the underlying fault. Calculated elastic deformations by dead load and post-tensioning of block 6 were smaller than the expected shrinkage gaps at the shear keys and it was decided to provide shear keys also at joint 6-6A, so that if necessary part of the horizontal load on block 6A can be carried over to the adjoining blocks. Furthermore, all contraction joints between blocks will be grouted up during this winter.

On the near-vertical rock-concrete faces a special contact grouting was carried out and a fan-shaped pattern of grout holes was used to connect the upstream grout curtains of blocks 6 and 6A.

The total length of blanket grout holes was 2,366 lin. ft. and its take 545 bags of cement. The curtain grout holes, total length 6.327 lin. ft., were stage grouted at pressure of approximately 1 lb./sq. in. per foot of depth and its total take was 1,126 bags of cement, including wastage.

The floating gate has five flooding compartments of total 3,150 cu. ft. volume. With empty ballast tanks the gate floats on its belly 10° off the horizontal and has a draught of 4 ft. By flooding the two lower ballast tanks the gate turns to a near vertical position with a draught of 12 ft. The other three tanks allow [illegible]rate positioning in the dam recesses provided. The positioning of the gate on the dam was carried out recently without any difficulty. For the floating gate a mooring jetty was constructed near the dam, where the gate will be moored in its vertical position with a draught of approximately 14 ft.

Adjacent to the jetty a slipway has been constructed on which the gate can be completely hauled out of the water for painting and other maintenance.

The seal between the gate and the side pier plates is obtained by a $\frac{5}{8}$ in. thick rubber sealstrip bolted to the gate. These seals overlap the gate edge by $2\frac{3}{4}$ in. and unrestrained angle up at 60°.

The horizontal crest seal is a $6\frac{1}{2}$ in. x $\frac{1}{2}$ in. flat rubber, bolted to the gate and to dam crest, with the hinge pivot line going through the centre of the seal.

Around the hinge bonnets $\frac{1}{2}$ in. rubber stub seals are used with rawhide gaskets formed around the bonnets with $1\frac{1}{2}$-in. upstands, both bolted to a steel frame set in the concrete dam crest and pressed against the hinge bonnets.

Some minor leakage occurs where the horizontal crest seals meet the side seals and also around the hinge bonnets, but the total leakage is at the moment much smaller than the half cusecs allowed in the design specification for the flap gates.

The test results on hard grade steel bars to which spacer bars were welded in the laboratory can be summarised as follows.—

1. Yield points were not significantly altered by either welding or heat treatment.
2. The average ultimate tensile strength of the bar was affected considerably by welding, approximately 25 per cent loss of the strength of the unwelded bar.
3. Normalising restored the UTS on the average to about 95 per cent of the original strength.
4. Elongation fell considerably in the welded bar but was nearly completely restored by heat treatment.
5. The fatigue strength was lowered by an average of 30 per cent; heat treatment restored only part of the lost fatigue strength. Some of the welds on the actual bars in the dam showed on inspection bad undercut which would explain the breaking of one bar by the slip of a jack hammer. As the welding had not been recorded when carried out, it was doubtful that no welding had occurred to the bars deeper in the concrete.

Uncovering of weld locations for heat treatment by breaking out the surrounding concrete proved impracticable, due to the confined working space in the hinge blockouts and the damage occurring to the anchor bars by the slippage of the jackhammers.

As all dam works were carried out by contract, no reliable unit rates can be provided. The total cost of the dam was £1,800,000 of which the contract for the five flap gates was £289,000 and for the three sluice gates £34,000.

To Dr. Carter.—The additional comments given on the history of the geological investigations carried out and the geology of the Scrivener Dam site in particular are welcomed and greatly appreciated and answer some of the queries raised in the discussion without further explanations of the authors.

Bridges in the Canberra Central Lake Area—Design

BY E. M. BIRKETT, B.Sc. (*Member*) and G. N. FERNIE, B.E. (*Associate Member*)*

Summary.—A number of prestressed concrete bridges of some interest have been developed and constructed simultaneously with the Lake Schen during the past five years. This Paper describes design features, both structural and visual, of these works, giving particular attention to the two ma bridges over the Lake at Commonwealth and Kings Avenues.

Introduction.

Rarely are two major bridges the subject of design and construction in a single scene and within the same space of time. Notwithstanding the quality and architectural imagination which has, consequent to their position, been devoted to them, the cost of these structures is, if anything, less than for bridges of similar magnitude in less demanding settings. That this is so is in large part attributable to recent advances in the design and construction of precast prestressed concrete as a structural medium, and of course to the keenness and efficiency of the Australian contracting industry.

A description of the visual considerations taken into account by the Consulting Engineers and their architectural colleagues, William Holford & Partners, is followed by a dissertation on the engineering features of the two main bridges. In both structures precast units were used: T-beam sections in the case of Kings Avenue Bridge and 50-ton 10 ft. long box sections in Commonwealth Avenue Bridge. Reference is also made to a number of smaller bridges, some of which used precast beams and others were of *in-situ* construction. In all cases prestressing was used.

The Bridges in Their Setting.

During past years the absolute symmetry of Walter Burley Griffin's inspired plan for the National Capital has been amended to a more informal and, from a traffic point of view, more practical arrangement. The main features still hold and important amongst these are the two main traffic lines on the flanks of the Triangle linking the gap between the two parts of Canberra formed by the Molonglo floodplain.

To assist further in this conception of uniting the setting, the bridges at Commonwealth and Kings Avenues have both been developed as twin bridges in alignment with the existing dual roadways and their broad median strips. Arising from this arrangement is the added advantage that the consequent space between the bridges permits the addition of further roadway capacity or some other form of transport if the future so demands.

The feature of the landscape that has had the most influence on the design of the bridges is the lowness of the relief in relation to surface distance. This characteristic—which is not at all the same thing as flatness—is the rule in Canberra; the hills are in the middle distance, the mountains in the far distance. For the most part, the attenuated undulations of the landscape are apparent in lines that lie close above one another in the region of the horizon.

Because of the width of the natural flood plain, horizontal lines control the form of the bridges. Needing little clearance and low abutments, the bridges take an appearance sympathetic to their background without particular effort on the part of the designers.

Also of importance and further emphasised by the Lake is the exceptional clarity of the atmosphere, thus minimising the effect of distance, so that far-off objects look deceptively near.

In both bridges the spring of the structure from bank to ban has been clearly defined. It was felt that any suggestion of arcuatio of individual spans would detract from the effect. At Kings Avenu the logical adoption, from the required clearances and profiles, a difference in curvature of upper and lower flanges provides th character of the bridge. Commonwealth Avenue Bridge, being landmark from many viewpoints, needed a design strong enoug to hold its own in distance as well as in close views and a parall section in a flat but distinct vertical curve was adopted.

It is in the treatment of the piers that lies the main differenc between the appearance of the two bridges. At Kings Avenu the bridge closes the Central Basin from the less interesting Ea Basin and has the piers of its twin structures so that, when seen i echelon, they form a wall permitting only narrow glimpses of th basin beyond. At Commonwealth Avenue, the opposite condition prevail. The view is closed naturally by the Acton Peninsula an Black Mountain and the opening between the West and Centr Basins needs to be maintained. The buildings on the shore c the West Basin will be of interest and Commonwealth Avenue is frame to views above and below it. The piers therefore have bee made narrow and widely spaced.

The characteristics of the bridges have been further define by the design of the lighting and at night time both bridges wi show up clearly as single structures sweeping from bank to bank

These have been the main considerations which, in compan with the engineering design criteria, have resulted in the structure as designed and built.

Design Criteria.

(*a*) *Loading:*

All road bridges were designed for H20-S16-44 loading i accordance with the N.A.S.R.A.A. Highway Bridge Design Specifi cation (1958).

(*b*) *Stresses:*

The following codes of practice were generally used for th determination of allowable stresses.—

Reinforced Concrete: B.S. C.P.114 (1957)
Prestressed Concrete: B.S. C.P.115 (1959)
(The draft Australian Code of Practice for Prestressed Concrete ha not been published at the time the major design work was undertaken.

Concrete with minimum cylinder compressive stresses o 6,000 lb./sq. in. at 28 days was specified for prestressed work excep for the segmental jointing of Commonwealth Avenue Bridge wher 6,500 lb./sq. in. was called for.

(*c*) *Model Tests:*

Results of model tests on the Hammersmith Flyover, London and the Narrows Bridge, Perth, were of use in the design of Commonwealth Avenue Bridge (Refs. 1, 2); otherwise no structural mode testing was employed for the structures. However, the Commonwealth Department of Works carried out during 1959 and 1960 a considerable number of hydraulic model tests which assisted in th design of the two major bridge waterways and embankments.

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Mr. Birkett is the Australian partner of G. Maunsell & Partners, Consulting Engineers, and Mr. Fernie is the firm's Engineer resident in Canberra.

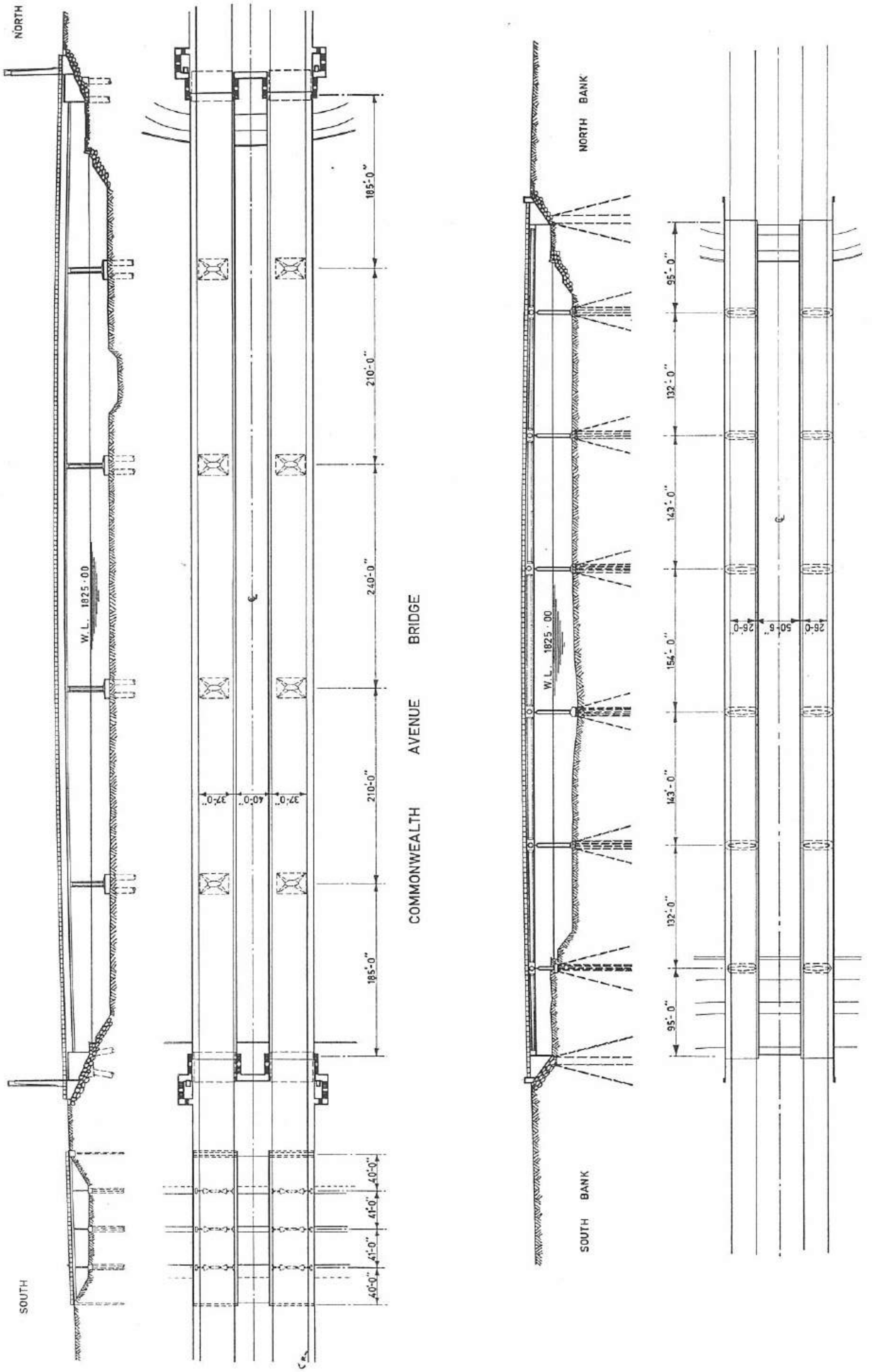


Fig. 1.—*Kings and Commonwealth Avenue Bridges—Plan and Elevation.*

Kings Avenue Bridge.

As mentioned earlier, the bridge is constructed as two separate roadways which are each of two lanes in width, separated by a gap of 44 ft. It has been designed as a 7-span prestressed concrete girder deck with the spans graduated into lengths of 95 ft., 132 ft., 143 ft., and the central span 154 ft. This allowed the provision of underpass roads on both shores (Fig. 1).

The T-beam deck is carried on slim precast concrete piers founded upon steel tube piles 18 in. in diameter.

Peak flood flow characteristics through the promontories were improved by the excavation of a channel 750 ft. long, 300 ft. wide, and 20 ft. deep beneath the bridges.

Site Conditions:

The River Molonglo in its original position meandered across the plane area of the bridge. Although the flow in the stream is normally low, the liability of the river to severe flash floods made it desirable that the design should be such that the methods of construction should not involve substantial temporary works in the flood waterway.

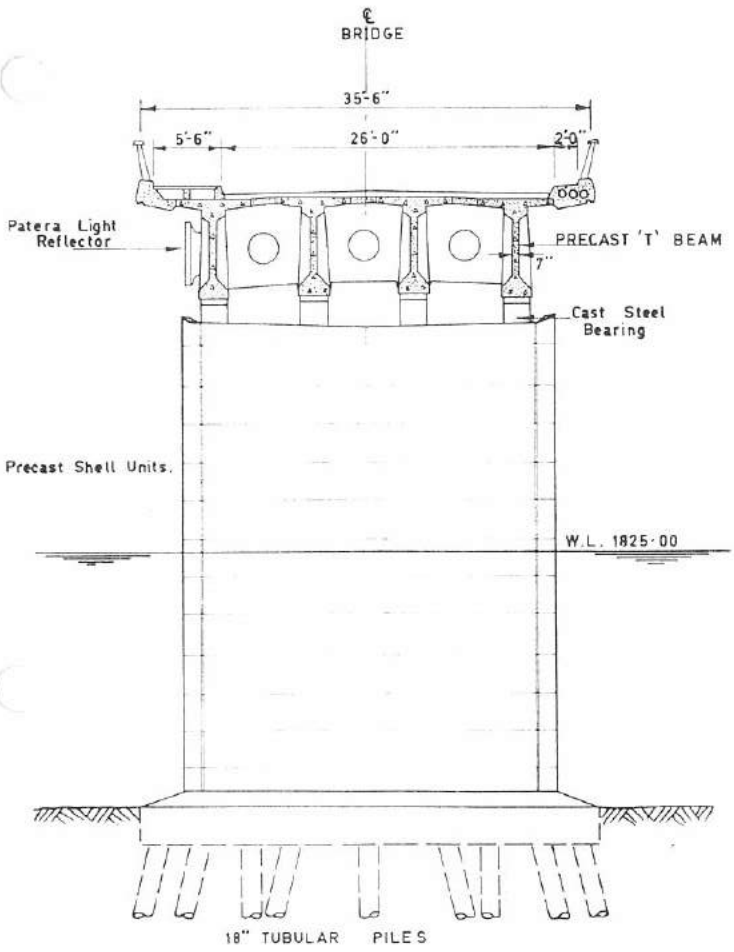


Fig. 2.—Kings Avenue Bridge—Cross Section.

The site is covered by the alluvial deposits of the Molonglo; heterogeneous sands and soft silts, of depths varying from nothing to 30 ft. overlaid a stratum of sand or gravel generally about 7 ft. thick. Below these recent strata lay shaley clays which are of varying hardness but generally become harder with depth. It was envisaged that no difficulty would be encountered in the ability of this material to provide supports for piles driven into it.

Substructure:

The bridge was designed around the use of steel tubular piles 18 in. in diameter, the steel tube being fitted with a toe and driven from the head. Once the set had been reached, the pile would be filled with concrete reinforcement in its upper lengths. Some difficulties were encountered in the driving of these piles and a number were driven open-ended. The steel piles were preferred to prestressed concrete piles as they were thought to be easier to drive and less affected by the hard driving. In view of the uncertainty as to the length of the piles, the steel piles can be more conveniently made and extended as required.

The general form of the piers reflects the nature of the forces to which they are subject: substantial wind and water forces in a direction transverse to the bridge and less severe longitudinal forces due to bearing friction and nominal water action. They were designed of a streamlined section in order to reduce obstruction of the waterway to a minimum. For ease of construction and for the twin objects of producing a high quality surface to resist the erosive power of the river in flood and to provide a good surface for the piers, the design incorporated 4 in. thick precast concrete elements set one upon the others; the strength of the pier being provided by reinforced concrete heartening placed in the space enclosed by the precast shells.

The abutments were designed in *in-situ* concrete to provide support for the shoreward ends of the bridge in a manner substantially independent of the approach embankments. These were designed to end in a free slope so that the abutment masks rather than retains the material.

Again steel tube piles were used for the foundations and the outer wing walls and intermediate diaphragms cantilevered back from the main base slab. Blankets of rock fill have been placed in both embankments where they spill around the abutments to form the river banks in order to reduce possible erosion.

A simple architectural arrangement of shallow recesses was provided in the otherwise long blank faces of the abutment walls.

Superstructure:

In contrast to Commonwealth Avenue Bridge, the Kings Avenue Bridge deck consists of a grillage system using four longitudinal precast prestressed concrete T-beams connected by transverse *in-situ* reinforced concrete diaphragms (Fig. 2).

The bridge elevation shows the beam depths varying so as to produce a slight bowed effect of the upper chord line relative to the lower. This variation is the natural outcome of the increasing span lengths towards the bridge centre. The beam depth at the abutment is 5 ft. 3 in. and at the centre 8 ft. 3 in.

Individual spans were simply supported under all but a small proportion of dead load. Under live loads full continuity, achieved by the placement of high tensile bars in the *in-situ* deck concrete over piers, resulted in a reduction of mid-span moments.

Halved anchor blocks were used for economy of pier width.

The erection sequence required to produce this articulation is as follows.—

1. Beams stressed and launched.
2. Diaphragms cast except over piers.
3. *In-situ* concrete cast between T-beam flanges.
4. Footways cast (the stresses induced are low enough to allow flexibility in sequence of this operation).
5. Pier diaphragms and deck concrete over piers cast, including steel, making structures fully continuous.
6. Remainder of dead loads added.

The construction of span diaphragms linking the four beams prior to footway casting allowed redistribution of this footway loading.

Bending moment distribution in the deck grillage was analysed by the methods of Morice and Little (Ref. 3). To allow for continuity the effective span was taken between the points of inflection given by the total moment, a satisfactory assumption in a no-torsion grillage for longitudinal moments. For transverse moments the actual span length must be taken because of the assumption of constant transverse deflection profiles.

As local wheel load moments and shrinkage will in theory ·end to crack the deck slabs, this assumption of no-torsion is reasonble. In the case of the transverse diaphragm section properties, ·oth the cracked and uncracked cases were found to have no signifiant effect on this assumption. Transverse shear and moments for nis grillage were in fact very low, in relation to local wheel load ffects on the slab. The highest proportion of two-lane loading ıken by each beam in turn across the section (Fig. 2) was 0.245, .26, 0.29 and 0.35.

Use was made of the Ferranti "Pegasus" computer, first to alculate the section properties together with fixed end moment ıctors for the individual spans (Ref. 4) and, secondly, to obtain ending moment influence lines for the continuous structure. The ıtter operation involved rapid solution of the eight simultaneous quations evolved from slope deflection theory for five influence ›ading points taken separately in each span.

'restressing :

Longitudinal prestressing design incorporated 12/0.276 in. ia. wire cables pulled to a specified 130,000 lb. and anchored by reyssinet cones. Mid-span design losses for the 154 ft. span were 6 per cent including 10 per cent friction. The initial specified ›ad did not include anchorage slip which was additional to it, epe·dent on field results. The number of cables in the beams ari rom nine cables in the short end spans to seventeen in the :ntre span.

The methods of anchor block analysis of Magnel and Guyon ere the subject of investigation by S. P. Christodououlides in 1955, ho showed that in certain cases these methods under-estimated ıchor block bursting stresses by as much as 200 per cent. Tests y the U.K. Cement and Concrete Association (Ref. 5) undertaken the light of this work on individual anchorage stresses had not ›mmenced at the time of the Kings Avenue Bridge anchor block :sign.

It is felt by the authors that shape is of prime importance in ıy multiple anchorage block design. In Kings Avenue Bridge, here adequate length of block is practicable and smooth lead-in ›nes from the load concentrations to the web and flanges are ·ovided, it is considered that Guyon's standard end-block stress ıalysis for reinforcement design is adequate.

However, where sharp changes of section occur with shallow locks, it is necessary to consider tensile stresses developed by disırtion of the block. Again, where particularly high anchorage ›ncentrations occur, it is advisable to use more conservative ıethods such as those of the Cement and Concrete Association.

lr ectural Features :

Sandblasted aluminium posts and railing were designed to ·ork in conjunction with dark anodized aluminium hoods.

Strip fluorescent lighting was provided in the outer handrail ood of each structure. This was intended only to light the footath but did in fact result in carriageway illumination as well, ıus leading to its more extensive application later at Commonwealth ·venue Bridge.

Five foot diameter metal reflectors placed over the anchor lock halvings at the piers are lit by spot lights placed above them ı the footway soffit. These lights in addition are directed onto ıe outer pier edges.

'ost :

The cost of this structure was £11 10s. 0d. per square foot xcluding surfacing with a ratio of foundation (including piers and butments) to deck cost of 1 : 1.

Commonwealth Avenue Bridge.

This bridge is made up of twin structures spanning 1,020 ft. etween abutments. On the south bank dual four-span underpass :ructures each 162 ft. in length allow provision for future cloverleaf ›ad approaches (Fig. 1.)

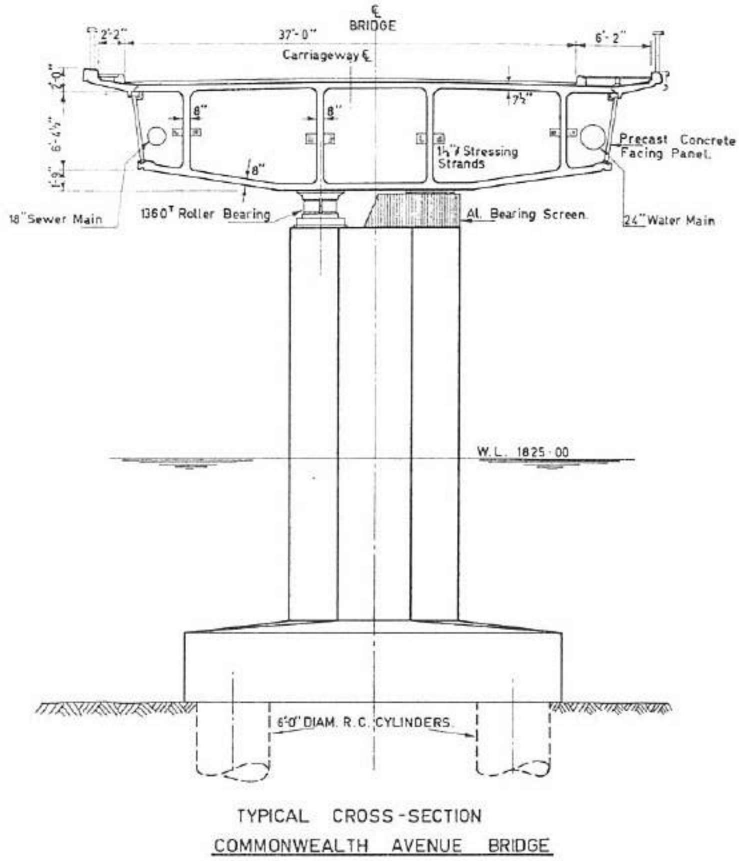


Fig. 3.—Commonwealth Avenue Bridge—Cross Section.

The main superstructure is of constant multi-webbed box section shape, continuous over five spans in varying lengths of 185, 210 and 240 ft. Slim octagonal central piers are carried generally on 6 ft. dia. reinforced concrete cylinders (Fig. 3).

Distinctive lines to the structure are provided by the upward sloping beam soffit combined with carefully chosen architectural treatment.

Site Conditions :

Sandy to silty clay strata overlie gravel deposits beneath which are two main rock types. Hard limestone occurs immediately beneath the alluvium to the south side. Over the rest of the bridge line siltstone deposits are found, the upper layers of which are decomposed to varying degrees.

The north embankment is cut by the Molonglo River which can infrequently flood the otherwise dry lengths of the bridge to a depth of several feet. From a design viewpoint, the bridge could be regarded as being built in the dry.

Substructure :

The bridge was initially designed to be carried on large spread footings except at two pier locations where piles similar to those used on Kings Avenue Bridge were proposed. As the result of an application by the Contractor the foundations were redesigned on the basis of large bored cylinders.

Triaxial testing showed the decomposed siltstone to be a frictional material having in many cases a high cohesive strength. Typical founding level values were a friction angle of 20° and a cohesion of 2,000 lb./sq. ft. Maximum cylinder bearing loads of 10 ton/sq. ft. ignoring side friction were taken for this material, mainly on the basis of desirable settlement limitation.

Laboratory consolidation tests showed moderate compressibility of both a rapid and reasonably consistent nature. It was appreciated that the massively bedded *in-situ* material would probably exhibit much more favourable behaviour than that predicted by tests on 2½ in. dia. core samples. Nevertheless settlements

in the order of 2 in. were calculated from these tests and used in assessing the overall structural strength of the bridge.

Cylinder rakes of 1 in 10 maximum were adopted for the pier foundation. At the south abutment where rock is encountered only 20 ft. from the surface, rakes of 1 in 6 were found to be economical for resisting longitudinal forces of 180 tons which would arise from bearing fixity at that end acting simultaneously with earth fill pressures.

The standard pier design was a four-cylinder solution with a maximum cylinder load of 850 tons. Provision was made in this design for the incorporation of two extra cylinders where decomposed siltstone occurred, thus reducing the maximum cylinder load to 600 tons. The use of 9 ft. dia. bell ends to the cylinders reduced maximum bearing pressures to less than 10 ton/sq. ft. in the decomposed siltstone. A design pressure of 25 ton/sq. ft. was used for rock-based foundations.

Normal reinforced concrete design was employed for cylinder caps and columns, $1\frac{1}{4}$-in. square twist steel working to 30,000 lb./sq. in. under the most severe loading combination being used in the cylinder caps.

Cylinder fixity was provided for in the cap, and the cylinder reinforcement generally extended down to within 5 ft. of the toe, the design being based on the assumption that the cylinders were generally pin-ended. Where 9 ft. dia. bell ends were employed on relatively unyielding material then clearly some fixity at the base would occur. However, the concrete tensile stresses resulting would be reduced to acceptably low figures by the vertical cylinder loading. The consequences of cracking, if it did occur under exceptional loading, were not considered to be in any way serious.

Designed on the basis of standard reinforced concrete cellular construction, the abutments house the expansion loops and bends for all main pipe services.

The vertical load of 730 tons from the anchor block bearings was designed to be taken down to the cylinders by the 4 ft. thick front wall spanning 40 ft.

Diagonal shrinkage cracking occurred across these walls, due apparently to their massive nature and the restraint to movement provided by cylinder rigidity. In future designs of this type thinner walls with shear steel designed to take all vertical loading would be used.

It should be noted that in this case it was clear that arch action relying on the heavy longitudinal bottom steel in the 15 ft. high wall dealt quite satisfactorily with the loading.

Roller Bearings:

The bearings used on this structure are of particular interest. Manufactured by Fritz Kreutz of Dusseldorf, they have undergone a special case hardening process which allows working Hertzian bearing pressure of 160 tons/sq. in. Each pier roller bearing is designed for 1,360 tons and yet is only $9\frac{7}{16}$ in. dia. and 2 ft. 10 in. long. Had the same diameter bearings been designed to the highest allowable line loads in B.S.153 a length of 17 ft. would have been required.

Standard German practice for these types of rollers is to adopt a friction coefficient of 0.03, and this figure was used in the design. Nevertheless tests on the same roller type of English manufacture gave a figure of only 0.002, a figure confirmed by longitudinal jacking of this bridge.

Each roller bearing can be altered vertically by ± 2 in., allowing compensation for pier settlements to be undertaken by jacking under the diaphragms and inserting or removing packing as required. The superstructure detail incorporated provision for jacking the whole structure longitudinally to compensate for differences from design longitudinal bearing movements obtained in the field after stressing. Such provision was considered essential for structures of this length stressed in one operation.

Superstructure:

Adoption of a fully continuous design with only one anchor block at either end of the 1,020 ft. long " beam " was made after four primary considerations.—

1. Excellent falsework foundation conditions and the provision of temporary stressing of beam sections.
2. Elimination of massive intermediate anchor blocks and strand anchorages.
3. Elimination of large and uncertain creep-time vertical deflections inherent in cantilever construction which could ruin the strong parallel beam curvature.
4. Assistance provided by secondary prestressing moments described below.

The box section chosen has considerable torsional strength, and in line with modern bridge practice is able to carry live loads eccentric in the transverse direction without any part of the beam being disproportionately stressed in bending due to that eccentricity.

In this case then, the total bending strength of the box section is closely equated to the maximum applied bending load and no superfluous strength is provided.

For design purposes the small proportion of *in-situ* concrete in the footways and cable boxes is ignored in the assessment of longitudinal stress.

The structure is fully continuous under dead and live loads. Secondary moments induced by the redundant reactions during stressing have an important modifying effect upon the resultant prestressing moment (Table I and Fig. 4). The resultant prestressing moment induced is of larger value at interior supports and correspondingly reduced over mid-span than would be the case if the beams were not continuous. Choice of the cable profile used this effect to balance out peak working load moments, enabling adoption of a constant cross section throughout.

Calculation of longitudinal bending moments and influence lines involved the solution of a continuous beam structure four times statically indeterminate. The problem was conveniently solved through linear analysis by evaluating the influence coefficients for the indeterminacies. This work was carried out by the " Pegasus " electronic computer using a matrix algebra programme.

TABLE I.

Commonwealth Avenue Bridge—Longitudinal Bending Moments.

Total bridge moments (ft. tons)	Centreline 185 ft. span	Pier 1	Centreline 210 ft. span	Pier 2	Centreline 240 ft. span
Beam full dead load	+17,800	−26,050	+9,950	−30,050	+19,350
Beam live load ((Sagging) + ve)	+ 5,150	0	+4,800	0	+ 5,850
Beam live load ((Hogging) − ve)	− 1,600	− 6,050	−2,550	− 7,000	− 2,000
Secondary prestressing	+ 2,400	+ 4,650	+4,700	+ 4,700	+ 4,700

Transversely prestressed diaphragms occur over the piers and at approximately the third points in every span. Over the piers they transfer outer web loads back to the roller bearings in addition to accommodating an 11° cable direction change. Intermediate diaphragms ensure no relative web distortion at the two changes of cable direction in each span.

The precast box units were designed as Vierendeel girders 9 ft.-9 in. long and weighing 50 tons; these units can be supported during erection at either the inner or outer edges. Square torsion twisted bars used for primary transverse steel work up to 31,000 lb./sq. in. Crack widths at this stress were estimated at 0.006 in. by the " Swedish Tentative Specifications for Limitation of Crack Widths in R.C. Structures ". Cracks slightly less than this occurred during erection.

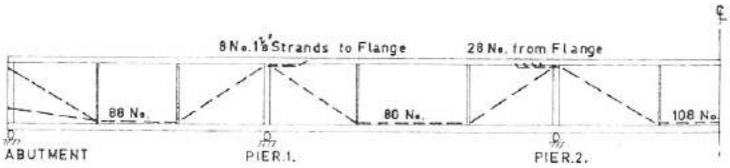


Fig. 4.—Commonwealth Avenue Bridge—Longitudinal Stressing Profile.

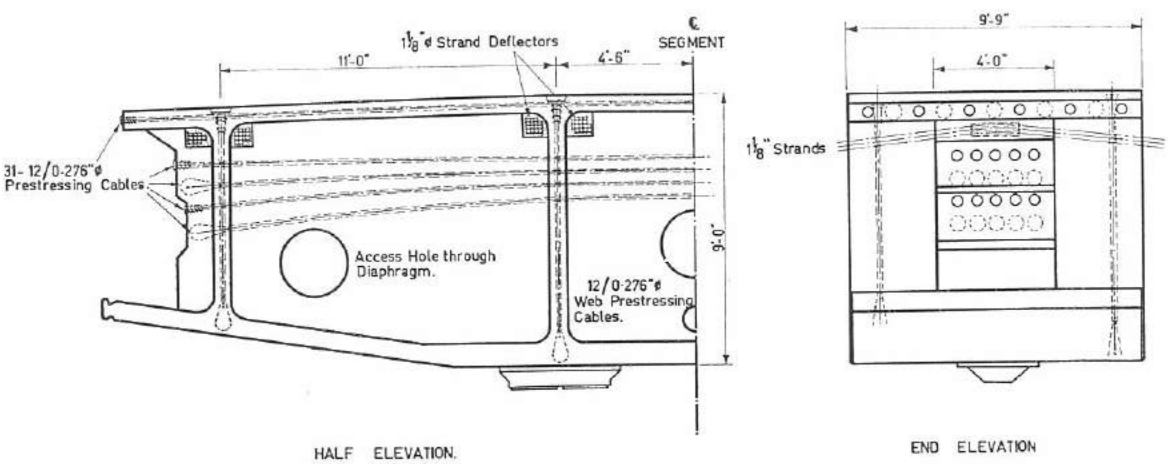


Fig. 5.—Commonwealth Avenue Bridge—Transverse Pier Diaphram.

Immediately prior to jointing and stressing these box units needed to be supported in the " no sway " position under each web. This ensured that the super-position of axle loads did not overstress the box units transversely. A margin of bending strength remained in the transverse direction to accommodate variations from the ideal " no sway " support condition which would occur in practice.

Prestressing :

Nineteen-wire 1⅛ in. dia. strands placed in groups of up to 15 either side of the webs provide a maximum of 6,000 tons of longitudinal prestress in each bridge. With a specified minimum ultimate of 181,000 lb. initial loads of up to 145,000 lb. were used requiring a 7-ft. extension in these 1,050 ft. long strands. This entailed three pulls from both ends requiring three separate anchoring operations with specially tested Gifford-Burrow wedge and barrel anchorages.

Prior testing had established anchorage efficiencies as low as 90 per cent which meant that under exceptional circumstances initial prestress could be as high as 88 per cent of the system's ultimate, a load which might well be considered undesirable. The inclusion of four extra strands in each bridge allowed for in the saddles and anchor blocks arrangement to cover variations in initial design assumptions enabled the actual initial load used on the bulk of the strands to be reduced to 142,000 lb.

Vertical 12/0.276 in. dia. wire cables with looped dead end anchorages were used in the box webs adjacent to piers where they reduced principal tensile stresses to less than 175 lb./sq. in.

The pier diaphragms each have 1,360 tons of this same type of transverse prestress (Fig. 5) applied in two stages. Prior to jointing and longitudinal stressing, 16 lower cables pulled to 44 tons each induce compressive stresses in the isolated I-beam unit necessary to control principal tensile stresses in the 3 ft.-6 in. thick webs.

Longitudinal continuity of the deck is then obtained by pulling twenty 1⅛-in. strands. Stressing the remaining 15 transverse cables follows in the second stage.

Thus the second stage eccentric cable loads disperse in the upper and lower flanges over a wider and less specific length either side of the diaphragm, avoiding any sharp stress discontinuity in these flanges.

Differential shrinkage cracking occurred prior to stressing of these diaphragm units resulting primarily from the restraint provided by the well reinforced flanges. These resulting web cracks did not appear to close up completely on stressing but subsequent test loading to full working load showed the diaphragm behaviour to be quite normal. Nevertheless, had the design been based on casting and stressing the central web initially as a rectangular " biscuit " independent from the adjacent flange and box section this cracking would have been overcome.

The design adopted for anchor blocks represented the first application of the Cement and Concrete Association Research Pamphlet No. 9. Bursting steel is provided on the principle that the tensile force varied as the ratio of loaded anchorage area to concrete area around the anchorages. It was adopted for these specially shaped anchor blocks using successive resultant areas as applied for instance in Guyon's anchor block design.

Elimination of the anchorage slip and elastic shortening losses, reduction in creep losses, and a substantial reduction in relaxation loss was achieved by allowance in design for restressing strands 300 hours after anchoring off. Extensions of up to 7 in. were calculated for this restressing operation, this extension being made up by the insertion of shims behind the anchorage barrel which, on restressing, moves away from the anchor plate together with the wedges as one unit.

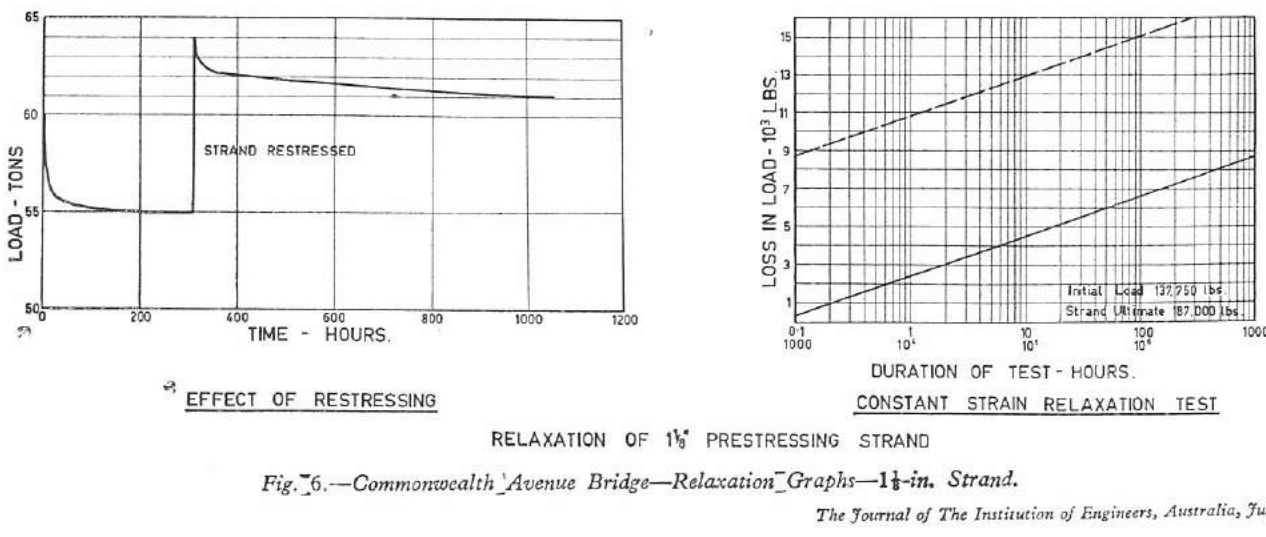
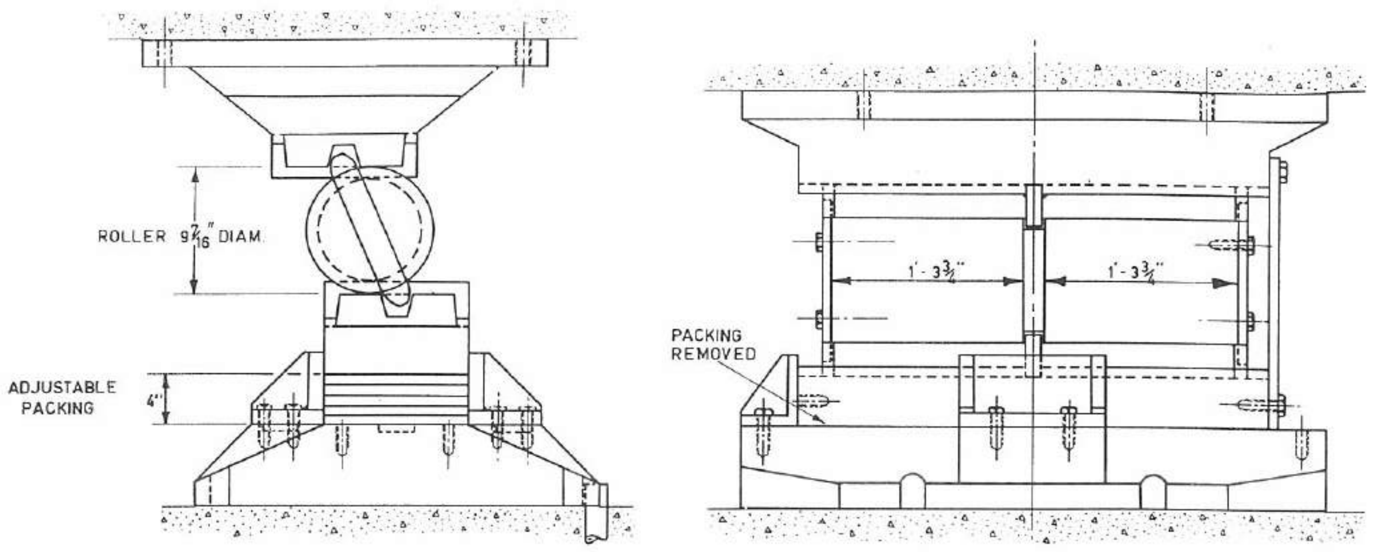


Fig. 6.—Commonwealth Avenue Bridge—Relaxation Graphs—1⅛-in. Strand.



SIDE ELEVATION ELEVATION

Fig. 7.—Commonwealth Avenue Bridge—1,360 Ton Adjustable Bearings.

Strand relaxation assessments deserve special mention. Extrapolation of the straight line relaxation loss versus log-time graph to the 100-year mark is a contentious and almost certainly conservative method of approach (Fig. 6). Tests conducted by Professor A. D. Ross for the Narrows Bridge, Perth (Ref. 6), have shown no measurable relaxation increase from 2 to 4 years using equipment of limited accuracy.

However, the relaxation loss over this period would only be 0.5 per cent of the strand load if the straight line graph held. It is still felt that highly refined testing is required over a much larger period to disprove the continuance of relation on a semi-log graph basis.

For this structure a typical ultimate relaxation loss was 15,000 lb. per strand at 100 years. Restressing at 300 hours then gave a nett design loss of 9,000 lb. or $6\frac{1}{2}$ per cent of the initial load in the strand at the particular point under consideration.

The following then are representative ultimate design losses, for a single strand after restressing to 141,500 lb.—Relaxation 9.000 lb., friction 11,300 lb. (at mid span), creep and shrinkage 00 lb., i.e., a total loss of 21 per cent representing a 10,300-lb. load gain per strand from restressing.

Structural Effects of Differential Settlement of Supports:

Two modes of pier settlement were examined: the first is uniform vertical movement of a pier as a whole, the second is unsymmetrical differential settlements involving horizontal longitudinal bearing movement or transverse movement allied to transverse tilt of the bearing plane. In each case the figures given below are for movements of Pier 2 relative to adjacent piers. Torsional shear stresses were calculated by Timoshenko's Membrane Analogy (Ref. 7).

A 1-in. instantaneous settlement induced 118 lb./sq. in. tension at Pier 3 reducing to 55 lb./sq. in. with creep relief. Under full design live load Pier 3 top and bottom flange stresses are +36 and +2,112 lb./sq. in. respectively. Thus the structure is relatively flexible in the vertical plane and under dead load conditions can accept gross movements.

The instantaneous tranverse pier tilt which would bring one bearing free of the superstructure is 1 in 436; after creep this tilt would be as much as 1 in 195. The structural effects of a tilt of 1 in 436 under full dead load with support on one roller only could be:

(*a*) Bearing load 1,608 tons as against design load of 1,360 tons, i.e., only 19 per cent overload.

(*b*) Inducement of torsional shear stress of 220 lb./sq. in. with a resultant 97 lb./sq. in. principal tensile stress.

(*c*) A transverse diaphragm bending moment of 6,510 tons ft. with a shear force 685 tons. Under this loading a principal tensile stress of 212 lb./sq. in. is induced in the diaphragm web.

In addition a 1-in. transverse horizontal movement at bearing level resulting from differential cylinder settlement would induce an immediate lateral reaction of 265 tons with ± 695 lb./sq. in. produced initially at the outer web reducing to 320 lb./sq. in. with creep. The minimum compression under full dead load at this point is normally 580 lb./sq. in.

Longitudinal foundation tilt would produce a horizontal movement at the pier top. A surplus of ± 2 in. remains in the roller over creep and temperature movements providing a generous allowance for such tilting.

Provision for bearing re-adjustment in addition to the order of stress given above is seen to make the structure well able to meet quite large foundation movements in the unlikely event of their occurrence.

Ultimate Load:

Superstructure mid-span and pier ultimate bending capacities were calculated by determining steel strains corresponding to a concrete strain of 0.3 per cent at rupture assuming plane sections remain plane. This method produced resultant total steel strains varying between 2 per cent and the ultimate of 4 per cent. However, the corresponding load increase for $1\frac{1}{8}$-in. strand between these strains, being less than 2 per cent, meant that a near balance in relative steel and concrete strengths was possible.

Without consideration of plastic hinge formation ultimate loads of 13 tons per foot run were then calculated.

However, tests on quarter scale models of the Perth Narrows Bridge in Southampton (Ref. 1) had shown that externally bonded strands produced excellent ultimate crack distribution in jointed segmental beams. Because of this sufficient beam rotation can be expected to provide plastic hinges without danger of local overstress.

The ultimate load was now calculated with hinges forming over the piers and at mid-span and this gave an ultimate load of 16 tons per foot run (Ref. 8). Because of the slightly over-reinforced condition and the nature of these assumptions a figure of 15 tons per foot was finally adopted corresponding to 1.75 (D.L. + L.L.).

Under normal support conditions with a ratio of dead to live load of 4 : 1 eccentric torsion loading would not reduce the bending ultimate.

Shear capacity was assessed by taking diagonal cracks of up to 60° from supports and providing sufficient steel to accept full load across those cracks.

As confidence in prestressed concrete continues it is the designers' opinion that the tendency will be more toward using part of the ultimate capacity for known exceptional highway loadings. In other words temporary cracking on infrequent occasions will be accepted in order to achieve further economies in the use of prestressed concrete.

Architectural Treatment:

The function of this bridge as the main link in the primary arm of the Parliamentary Triangle demanded a high quality of finishing detail.

Each of the four pylons rises to a height of 62 ft. providing a distinctive vertical emphasis and definition to the structure. They were designed as a thin tapering hollow reinforced concrete core having a white precast concrete fin and clad in Gosford granite panels.

Gosford granite and black Mudgee exposed-aggregate panels were specified for the abutments and central viewing terrace between bridges.

For the facing of the superstructure, white exposed-aggregate panels were again adopted, to give clean lines to the structure, at the same time protecting the services and cable concrete from direct contact with the weather. Special attention in detailing ensured that the panels could be easily removed and replaced by a simple cantilever cradle, truck-operated from the deck.

Anodized aluminium handrails (Fig. 8) house a continuous run of 4 ft. long 40-watt white fluorescent tubing pre-tested to illuminate the carriageway and handrailing itself. Identical lighting along the outer footway edges reflect off the white facing panels. In both cases care in design was taken to allow convenient maintenance access to the tubes by hinged aluminium covers in the case of the footway edges and removable hood sections 10 ft. long for the handrail.

At intervals along the bridge soffit groups of five 2 ft. long 40-watt green fluorescent tubing set in oval holes provide lighting for the waterway and piers.

Cost:

The cost of the main bridge was £10.5 per square foot excluding surfacing. The ratio of foundation to superstructure cost was 1 : 3.

Underpass Bridges.

These dual structures form a continuation of the Commonwealth Avenue Bridge road and walkway (Fig. 1). Their overall deck length is 162 ft. made up of two side spans of 40 ft. and two central spans of 41 ft.

Twenty-two inverted pretensioned T-section beams with reinforced concrete infilling form each simply supported slab deck span.

Pier crosshead supports are of inverted T-shape and the notched beams sit joggle fashion to present a flush soffit between main and cross beams.

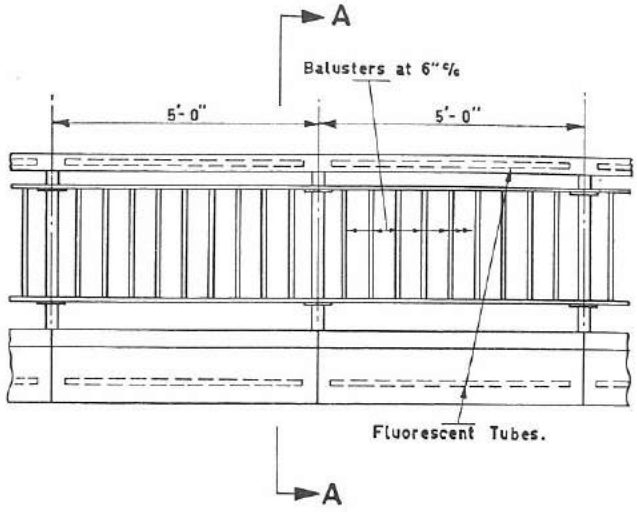
Duodecagonal columns with reinforced concrete hinges at the base are in turn supported on 18 in. dia. bored cylinders working at 45 tons each. Expansion movement at the north abutment is taken on rubber bearings.

The pretensional T-beam specification called only for top and bottom stresses of 0 and 2,000 lb./sq. in., respectively, allowing for all losses. In this way the choice of prestressing type was left to the individual manufacturer.

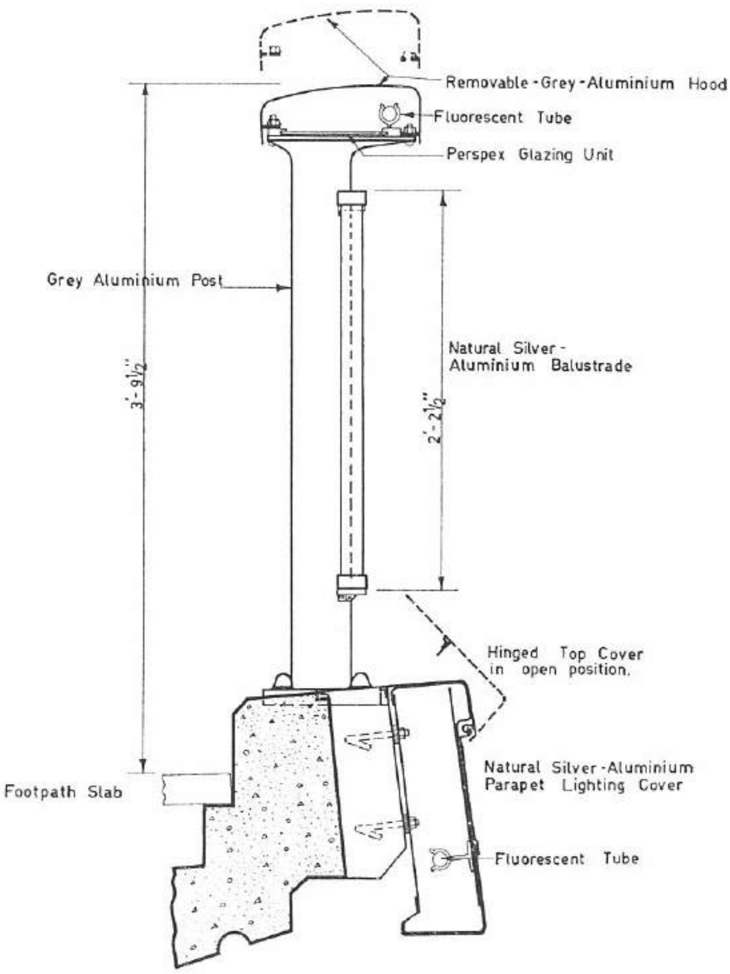
The structure excluding hotmix surfacing cost £4.5 per square foot.

Skew Slab Flyover.

Situated two hundred yards to the south-east of Kings Avenue Bridge the flyover consists of an *in-situ* post-tensioned concrete



ELEVATION
HANDRAIL & PARAPET LIGHTING COVERS



SECTION A-A.
HANDRAIL &
PARAPET LIGHTING COVERS

Fig. 8.—Commonwealth Avenue Bridge—Handrailing and Lighting.

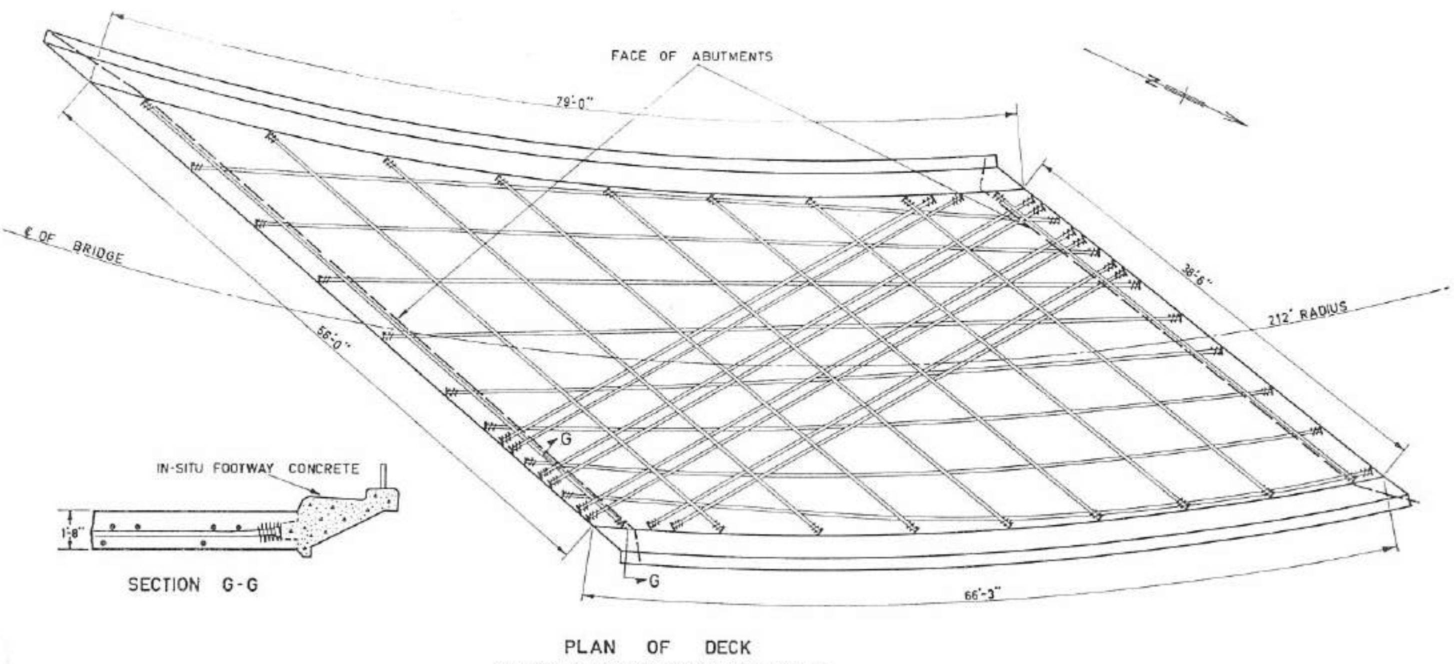


Fig. 9.—Skew Flyover—Plan Showing Slab Prestressing.

slab supported by reinforced concrete abutments. A 28 ft. wide carriageway is flanked on either side by footways two feet in width.

The geometry of the structure was unusually awkward. It formed part of a 212 ft. radius horizontal roadway curve with cross-falls of 1 in 12 and a vertical grade of 3.8 per cent. The curved edges formed skews with the abutments varying between 45° and 60°.

Deck Analysis:

The flyover has been designed as a flat elastic isotropic plate stressed in three directions along the lines of principal moment trajectories by twelve ½ in. diameter stranded cables (Fig. 9).

No rigorous analysis of this complicated deck form would be practicable except by means of a model test. In this case the size of the structure did not justify such a test. What has been done is to examine the plate in the light of theoretical and experimental work carried out on straight sided skew slabs.

Maximum bending moments under a uniform dead loading at free edge and centre were obtained for the 45° and 60° cases taken separately, with the footway edge beam loading approximated by taking a 2 ft. extra width of plate down each side.

In both skew cases there was little difference between the edge and central moments which were of the order of 78,000 and 55,000 ft. lb./ft. for the 45° and 60° skews respectively using a Poisson's ratio of 0.15 (Ref. 9). It was considered prudent to take the full 45° cases for the first analysis although the conditions on the western side would be more closely equal to the mean of both skew values.

The standard H20-S16-44 live load truck was then placed on the slab and central and edge deflection calculations obtained using the methods of the C. and C.A. Research Report No. 8 (Ref. 10). These were compared with the deflections from the uniform dead load cases and found to be slightly in excess of one quarter of these values both at the edge and centre.

The ratio of live to dead load moments was also found to be approximately of this same order. It was realised at this stage that as the live load stresses were only some 220 lb./sq. in. even wide divergence from the live load design approximations would not be critical to the structure.

Sufficient transverse prestress to take 60 per cent of the total longitudinal moment was supplied. This figure was certainly conservative.

The diagonal cables were placed across the corners of the abutments rather as a safeguard against heavy central loadings which would tend to span this shortest distance. Full St. Venant stress distribution from these more concentrated cables would be unlikely to occur across the whole slab width. They were considered rather as a type of "load balancing" system after the manner described by T. Y. Lin in a recent A.C.I. Journal (Ref. 11). The central cables give an upward lift to the slab across this diagonal band producing higher stress benefits at the centre of the slab than at the edges.

Initial Freyssinet jack forces as high as 85 per cent (374,000 lb.) of the specified cable minimum ultimate were used successfully on this structure.

"Metalastik" laminated rubber bearings were employed under the slab. On the high skewed south-western corner dowel bars were designed to prevent uplift. It is of interest to note that these bars were used over one third only of the abutment length. Instances have been recorded recently in Germany of serious moment cracking failure resulting from continuous restraint to uplift at the support edges.

Predicted central edge deflections of 3 in. under full dead load and creep calculated from the methods of Timoshenko and Jenson will, from measurements taken to date, be closely realised in practice.

Cost:

The deck slab itself without abutments, cost just under £4 per square foot.

Aquarium Pond Footbridge.

Aesthetic Considerations:

The close relationship with the water surface and flat surroundings determined both the structural form and style of the bridge (Fig. 10).

Originally a single span crossing was envisaged at this point. However, further consideration resulted in the unusual choice of a two-span structure. If the beam was to be built as a single span at a low level in harmony with its surroundings the space between beam and water would be no more than a horizontal slit. On the other hand to avoid emphasis on duality the success of the two-span design depended on the strength of the single arc achieved by the cantilever deck edge together with a reduction of the abutments to a visual minimum.

With so slender a deck structure, detraction from the fine white concrete parapet line was avoided by choosing anodized aluminium handrails in the lightest of grey colours, thus reducing their dominance in distant views. Advantage of reflections given by the siting of a weir just downstream of the structure was taken by providing a series of holes in the deck housing lights. These, in addition to casting light downwards for reflection off the water, also provided footway illumination.

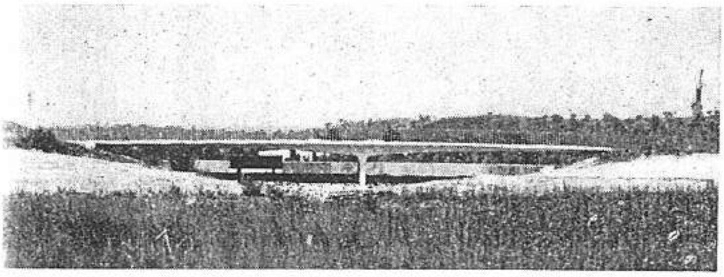


Fig. 10.—*View in Elevation of Aquarium Pond Footbridge.*

Design :

The design was of a two-span *in-situ* concrete beam continuous with the central pier and simply supported at the abutments. Stressing followed by using eight 12/0.276 in. dia. post-tensioned cables which by virtue of the strongly curving soffit at points of negative moment over the piers were able to run in a nearly horizontal line. Secondary prestressing moments, because of near cable concordance, were only $1\frac{1}{2}$ per cent of a total pier moment of 1.2×10^6 foot pounds.

Dead and live load moments were calculated by moment distribution after carry-over factors and fixed end moments were found by column analogy principles (Ref. 12).

Driven steel " H " piles of 87-lb. section were used at a 30-tons loading, under the pier and abutments.

Resonant Vibration :

In London, the St. James Park Footbridge was tested for resonant effects by marching guardsmen. For this structure the calculation of order of frequency likely to be met is of interest.

The bridge was analysed as being simply supported over the pier, for which condition the resonant vibration frequency for foot traffic is between 2.7 and 3.0 cycles per second. Under full loading the maximum strain energy of deformation for the first harmonic was equated to the maximum kinetic energy :

$$\frac{E\pi^2}{L^4}\int Ix \, . \, \sin^2\frac{\pi x}{L} \, . \, dx = \frac{4n^2}{g}\int \rho Ax \, . \, \sin^2\frac{\pi x}{L} \, . \, dx$$

where $L = 50$ ft., n = vibration frequency, ρAx = unit weight at x.

Each span was divided into five sections to complete the integration and the equation was then solved for n, which was found to be 2.36 cycles per second.

Thus, were the beams simply supported over the pier then the vibration frequency of 2.36 cycles per second may have been unsatisfactory. In practice the beam is simply supported at one end and virtually fixed at the pier. Even if the degree of fixity were as low as 70 per cent the natural vibration frequency would be greater than 3 cycles per second. From this it was concluded that the bridge would not exhibit resonant frequency under live load and this conclusion has been borne out by testing of the finished structure.

Acknowledgments.

The authors wish to thank Mr. J. W. Overall, C.B.E., M.C., F.R.I.B.A., F.R.A.I.A., F.A.P.I., A.M.T.P.I., Commissioner of the National Capital Development Commission for the opportunity to present this paper. They also express their indebtedness to the Consulting Engineers and their Associate Architect William Holford and Partners for permission to publish details of the design.

It is appropriate that they should associate with this work the Commission's Executive Engineer, Mr. C. J. Price, B.E., A.M.I.E. Aust., who has played a major role in the development of these structures.

Finally they feel attention should be drawn to the very high quality of workmanship and skill exercised by the Contractor, M. R. Hornibrook (N.S.W.) Pty. Ltd., in the execution of the work on the two main bridges under the direction of the Director of Works, Canberra.

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7. TIMOSHENKO, S. P.—*Strength of Materials*, Part II : Advanced Theory and Problems, 3rd ed. New York, Van Nostrand, 1956, 572 p.
8. LIN, T. Y.—*Design of Prestressed Concrete Structures.* New York, Wiley, 1955, *Chapter* 10, Continuous Beams, pp. 284-323.
9. TIMOSHENKO, S. P.—*Theory of Plates and Shells.* 2nd ed. New York, McGraw-Hill, 1959, 580 p.
10. ROBINSON, K. E.—The Behaviour of Simply Supported Skew Bridge Slabs Under Concentrated Loads, *op. cit.*, ref. No. 5, *Res. Report* No. 8, Nov., 1959.
11. LIN, T. Y.—Load-Balancing Method for Design and Analysis of Prestressed Concrete Structures. *Proc. A.C.I.*, Vol. 60, No. 6, June, 1963, pp. 719-41.
12. LIGHTFOOT, E.—*Moment Distribution ; a Rapid Method of Analysis for Rigid-jointed Structures.* London, Spon, 1961, Chapter 3.

Discussion

Mr. A. W. Knight (Member, Tasmania Division).—In opening the discussion I congratulate the authors for an interesting and instructive paper. The bridges discussed represent a high standard of work on the part of both the designers and builders. The critics have thus been set a difficult task.

It is accepted that these bridges, located as they are in a central position in a national capital, deserve special consideration in regard to their appearance, and it is evident from the paper that this aspect has received particular consideration. It is not often that the designer of a utilitarian structure is encouraged by the client to spend extra money for the sake of appearance.

Although it is clear that architectural aspects of the designs were given close consideration I am not altogether in accord with the results achieved. The adoption of twin bridges at both Kings Avenue and Commonwealth Avenue is commended both from the aesthetic aspect and because at some future time there may be need for wider traffic ways.

Designs involving clean simple lines are certainly called for on these sites but the camber incorporated in the Commonwealth Avenue Bridge is, in my opinion, excessive and detracts from the appearance of the bridge. The paper states that this bridge was designed as " continuous over five spans in lengths up to 240 ft. ". No justification is advanced for the adoption of such long spans on a site where construction of foundations was a simple matter. In my view adoption of seven spans would have resulted in a lighter and more economical structure more in keeping with the site. A structure has to be " right " from an engineering point of view to be " right " from the aesthetic aspect.

There are some statements in the paper regarding the design of the bridge substructures in relation to the views seen through the bridges. These arguments seem to me of little substance as from most positions the views of the bridges are in a downward direction and only swimmers are likely to see much between the piers of the bridges. To my mind the Kings Avenue Bridge is a more satisfying structure.

The selection of prestressed concrete as the medium for design of these structures is supported. For the sake of uniformity it might also have been adopted for the bridge structure over the dam which forms the lake.

The paper includes the statement " The box section has considerable torsional strength, and is able to carry live loads eccentric in the transverse direction without any part of the beam being disproportionately stressed ". The importance of this is worth emphasising. The merit of the box section has been appreciated and incorporated in several bridge structures in Tasmania.

I understand that some criticisms have been directed to the vertical pylons erected at the ends of the Commonwealth Avenue Bridge. This kind of ornament is not expensive in relation to the total cost of such structures and in my view is warranted.

In regard to costs I must say that I grew up in a period when bridge costs were very much lower than they are today. It is o[illegible] hase of engineering in which costs seem to have increased in [illegible] proportion to wage rates and little financial benefit appears to have resulted from improved techniques.

The design of the Aquarium Pond Footbridge has resulted in a very pleasing structure similar in principle to a bridge over the Rhone near Seyssel in France.

Mr. I. G. Cameron (Associate Member, Brisbane Division).—The authors are to be congratulated on their paper and on the quality of the bridges that their firm has designed in Canberra. Without wishing to detract in any way from the appearance of the bridges the writer draws attention to what he considers to be some misconceptions in the aesthetic analysis of the bridges as published by the authors. The Kings Avenue bridge, it is stated, has shorter spans and wider piers in order to block out the not so attractive East Basin view. Most people will view the bridge from the North (Civic Centre) Bank. As the view lines from this shore for about two-thirds of its length from the Commonwealth Avenue bridge only deviate $\pm 10°$ from a line normal to the Kings Avenue bridge most views will show the piers almost in end elevation. Therefore they will not form the visual barrier intended. Only when viewed at close range can the piers be seen in echelon. Nevertheless the spans selected for both bridges are [illegible]ng and fit the site well.

When viewed from a distance and at right angles to the bridge the camber in both bridges is pleasing. When viewed by the driver of a car the camber in the Commonwealth Avenue bridge is somewhat disturbing. The elevation of the crown of the Commonwealth Avenue bridge also prevents the vista down Commonwealth Avenue which was considered to be important in the original concept of the overall plan for Canberra. Were these aspects taken into consideration or was the camber fixed mainly on consideration of the external appearance of the bridge?

The cost of the two bridges has been given by the authors as:

Kings Avenue (ex bitumen surfacing)	£11 10s. 0d./sq. ft.
Commonwealth Avenue (ex bitumen surfacing)	£10 5s. 0d./sq. ft.

In August, 1961, at the Prestressed Concrete Symposium Mr. Birkett gave cost figures for the two bridges of £17 5s. 0d. and £15 10s. 0d. per square foot, respectively. Could the authors state the basis on which the now published costs have been calculated? What dimensions are used to calculate the area of the bridge and does the cost include engineering overheads?

The handrails on the Kings Avenue bridge have been canted outwards at quite a large angle from the vertical. This appears to have been done to enhance the external appearance of the bridge at night by making use of the lighting tubes in the top rail to light the external face of the bridge. The utility of the handrail seems to have been sacrificed to achieve this end. Now that the authors have seen the finished product would they again use this detail in another bridge?

The light reflectors on the Kings Avenue bridge girders are out of keeping with the rather severe classical line of the bridge and partly nullify the designer's intention of clearly defining the spring of the structure from bank to bank.

It can easily be demonstrated that external prestressing cables result in savings in first cost of the structure. However, most of the recorded failures in prestressed concrete structures have resulted from the corrosion of the prestressing wire where it has been protected by an unstressed covering of concrete. Do the authors feel confident that no trouble will be experienced with structures such as Commonwealth Avenue bridge and the Narrows bridge where such protection is used? It is cold comfort to know that inspection of the cables is easy as the problem remains as to what to do with the bridge if inspection reveals serious corrosion.

Mr. G. Fekete (Member, Sydney Division).—Mr. Fernie, in the oral presentation of his part of the paper, mentioned that he would like to see permission of some tensile stresses in prestressed concrete bridges of segmental construction, and also that a greater effort should be made to evaluate and to cater for shrinkage stresses. I agree with him on both counts.

Pipers Creek Bridge was constructed in 1953 on behalf of the Snowy Mountains Authority. I believe it was the first prestressed concrete bridge of segmental construction built in Australia, and I also believe that, with a span of 73 ft., it was at the time, and remained for a few years, the longest span prestressed concrete bridge in this country. In addition to normal highway loading, it was designed to carry the Authority's heavy load train consisting of two prime movers with a trailer of 40 tons tare weight and 85 tons payload. For normal highway loading no tensile stresses in the concrete were allowed, but a tensile stress of 200 lb./sq. in. was allowed for both initial tensioning and for passage of the heavy load train. The train with full load has been a number of times over the bridge, but no cracking has been observed.

The question of shrinkage is of course not new, and one may, perhaps, quote the concluding remarks from G. P. Manning's book, *Reinforced Concrete Design*, published in 1936:

> " At present designers pay great attention to calculating structural stresses, and, as a last thought, add a little steel at random to cover shrinkage and temperature stresses. As the shrinkage stresses are obviously often much higher than stresses due to loading, it would be more logical in many cases to make some attempt to calculate the shrinkage stresses and, as a last thought, add a little steel at random to cover structural stresses."

And finally, a question: How were the anchorage forces in the Commonwealth Avenue Bridge transferred to the slender webs?

The Authors in Reply:

Mr. Knight has emphasised two important problems of conception at Commonwealth Avenue Bridge, namely the choice of span lengths and the degree of vertical curvature.

In choosing the spans it was the designers' intention not only to maintain a visual relationship between the two bridges but to emphasise the senior role that Commonwealth Avenue will increasingly play. Had a 7-span solution been chosen, then a central span of about 180 ft. in relation to Kings 156 ft. would result, with an overall depth reduction of some 15 in. If it was economically reasonable it was felt that a central span, more important by international standards, was called for. It is the designers' contention that the relative costs per square foot of both bridges, i.e., £10.5 at Commonwealth and £11.5 at Kings suggests that this was obtained without substantial economic sacrifice. Despite this span increase, the overall external depth in elevation from parapet to soffit is an average of 9 ft. 0 in. over the centre span at Commonwealth, compared to 9 ft. 9 in. at Kings. The placing of white precast panels did in fact increase the apparent visual depth at Commonwealth.

Regarding the relative importance of the views through and beneath the bridges, the authors should point out that whilst many views are obtained above the general Commonwealth Avenue Bridge soffit level of R.L. 1848, the proposed construction of Parliament House and other national administrative buildings on the lower slopes adjacent to the Southern Lake Wall will place much greater emphasis on the impressions of horizontal views beneath the structures.

This same argument applies to *Mr. Cameron's* remarks on the effect of the Kings piers in closing off the east basin. The view from the Parliamentary Triangle is far more oblique and more close than that from the northern lake margin to which he refers.

One important point on piers for dual bridge structures is emphasised at Canberra. From certain angles it is difficult to determine just which piers belong to which side of each structure. Some improvement in future designs may be obtained by differentiation of piers by shape and colour.

Regarding the vertical camber at Commonwealth, two considerations arise: firstly, the need to maintain a 24-ft. vertical clearance for yachts beneath the centre span and, secondly, the need to reduce the cost and engineering difficulties of the embankments which on the north side are some 50 ft. in height resting on black silt with high settlement characteristics. It is agreed that any reduction of a humped appearance along the line of the avenue is to be aimed at. In this respect it is anticipated that traffic needs will produce pressures for an improved vertical alignment at the ends of Commonwealth Avenue itself. These in turn will reduce the visual impact of the hump.

Dealing with the query on costs, those quoted in the paper refer to the plan area of both the bridge and abutments. The previous figures quoted referred only to the bridge traffic lane width plus one half of the total footway width. Costs given in this paper are derived solely from the actual bridge construction costs given in the contract documents.

Canting outwards of the handrails at Kings, besides achieving lighting effects, gives an increased apparent width to the two lane structure. At Commonwealth, which is 50 per cent wider, vertical handrails were used. Where pedestrian usage is as infrequent as it is at Kings, this canted detail is not considered a disadvantage.

Mr. Cameron's remarks on external strand protection deserve special mention in Australia, where experience with it in conjunction with precast segmental construction is ahead of that elsewhere in the world.

It has been demonstrated exhaustively that cracks in reinforced concrete of less than 0.01 in. do not promote steel corrosion even under adverse exposure conditions. Again the presence of stressed steel *per se*, does not promote corrosion. If cracks of this size were serious then many equally important portions of reinforced concrete structures in addition to those necessarily unprestressed on a bridge such as Commonwealth Avenue would be in an unsatisfactory condition. This is particularly relevant in Australia where the progressive trends to concretes with higher shrinkage characteristics are creating serious problems.

The cable concrete at Commonwealth was a stiff vibrated mix, giving a $1\frac{1}{2}$ in. cover and placed quickly after stressing to take advantage of delayed creep effects in reducing crack sizes. Nevertheless shrinkage and bending cracking occur in the cable concrete and are at present some 0.005 in. wide.

It should be the prime aim of any design using external bonded cables to have ease of access. In this way should severe shrinkage cracking develop then prevention of corrosion can easily be achieved by a simple coating to affected areas. Inspection at 12-monthly intervals coupled with this must offer a complete safeguard against corrosion and can be achieved much more easily in a bridge box section than in many other types of reinforced concrete construction where it can be of equal importance.

The authors would indeed be interested to hear details of any prestressed structures which have shown signs of distress using these principles.

At the Narrows Bridge, conditions are less favourable. Ease of access is reduced by the I beam-diaphragm construction and a liquid 2 : 1 sand cement grout with high shrinkage characteristics was used. High shrinkage crack sizes developed and the concrete was subsequently coated with " Peratol ". It is hoped with the co-operation of the Main Roads Department, to expose a length of strand shortly and examine the steel strands over those sections where shrinkage was worst.

Mr. Fekete's comments are of real interest. It has been demonstrated on several of our segmental structures that bending tensile stresses of 300 lb./sq. in. produce no sign whatsoever of cracking at the joints. Using the techniques of an early temporary prestress as soon as 6 hours after pouring joint concrete against sandblasted precast unit faces, stresses of double this order (i.e. 600 lb./sq. in.) could be developed without obvious cracking. This would bring precast segmental construction into line with the thoughts on allowable tensions presented by W. Abeles* at the same time preserving the advantage of this type of construction in allowing higher stresses and avoiding shrinkage cracking; problems that must increasingly continue to exercise the designers of *in situ* prestressed concrete in Australia.

Regarding the anchorage query, the end blocks consisted of a 3 ft. thick " biscuit " extending across the full width of box with the 4 web sections tapering along a length of 8 ft. from 4 ft. wide at the " biscuit " to 12 in. wide adjacent to the standard box section.

*P. W. Abeles.—*Partial Prestressing in England.* Paper presented at Eighth Annual Convention of the Prestressed Concrete Institute.

REVIEW OF MAJOR DAMS IN AUSTRALIA

SCRIVENER DAM

NOMINATION FOR LISTING

ON THE

REGISTER OF THE NATIONAL ESTATE

Nomination prepared by

Mike Smith

and reviewed by

Lenore Coltheart

for

The Institution of Engineers, Australia

and the

Australian National Committee on Large Dams

January 1998

SCRIVENER DAM

NOMINATION FOR NATIONAL ESTATE LISTING

1. NAME, OWNERSHIP, LOCATION

Current Name	SCRIVENER DAM impounding LAKE BURLEY GRIFFIN
Previous Name	During and prior to construction, the dam was known as the CANBERRA LAKES DAM
Current Owner	COMMONWEALTH OF AUSTRALIA, administered by the NATIONAL CAPITAL AUTHORITY (formerly the National Capital Planning Authority). Address - 10-12 Brisbane Avenue Barton ACT 2600
Local Aboriginal Community ...	Ngunnawal
Location	The dam is located on the Molonglo River 5 km west of the Commonwealth Avenue axis through the city of Canberra
Map	The dam and its impounded lake are shown on the 1:25,000 CMA Map entitled "Canberra 8727-111-N" and on the 1:100,000 NatMap entitled "Canberra 8727"
Boundary	The boundary of the area (for National Estate listing) of the dam and its impounded Lake Burley Griffin should be that area encompassed by the normal top water level (TWL) of the lake plus the area of the dam including all appurtenant structures.
Local Government Area	The dam and its impounded lake are within the ACT government's local administration area but the land on which the dam and lake are contained are Commonwealth land under the control of the National Capital Authority

2. DESCRIPTION

Dam Type	Concrete gravity dam
Height	33 m
Length	235 m
Volume of Fill	54×10^3 m^3
Spillway Type	5 bays of free overflow crest controlled by hydraulically operated flap gates
Spillway Capacity	8,500 m^3/sec
Lake Volume	33,000 ML
Lake Area	6,640 ha
History	An ornamental lake along the Molonglo River was a key element of the original 1913 Walter Burley Griffin town plan for Canberra. However the dam was not constructed until the early 1960s.
Site Plan	A general arrangement drawing of the dam showing plan, elevation and sections is included in the attached illustrations.
Condition	The dam is in a reasonable condition, is well maintained and has not been modified during its lifetime. It is currently being subjected to a review due to criteria changes which may result in some modifications. It was originally built to impound a recreation lake and this has not changed.
Association	The dam was constructed in the period 1960 to 1963. The constructing authority was the former "National Capital Development Authority". The dam was designed by the then "Commonwealth Dept of Works" who also supervised the construction. The "Snowy Mountains Hydroelectric Authority" (SMHEA) was a sub-consultant for design of sluices through the dam. The dam was constructed under contract by "Citra Australia Ltd" and the spillway gates were detailed, supplied and installed by "A E Goodwin Ltd" in association with "Rheinstahl Union Bruckenbau" (West Germany).
Illustrations	Drawing:- Scrivener Dam - General Arrangement of Dam Colour photo-print:- Scrivener Dam in Flood, Nov 1991. *{Both included in Attachments}* Colour Transparencies:- *{Kodachrome 64 ASA daylight transparency film}* 1. Downstream View of Scrivener Dam 2. Upstream View of Scrivener Dam 3. Scrivener Dam Spillway Gate 4. View of Lake Burley Griffin from Black Mountain

3. STATEMENTS OF SIGNIFICANCE

Scrivener Dam and its impounded Lake Burley Griffin's claim for National Estate registration is primarily due to its relevance to the history of Canberra as Australia's national capital and some of the principal persons involved in its planning. An assessment of these claims relevant to the Australian Heritage Commissions criteria is given below.

The dam has already been nominated to the interim ACT Heritage Places Register under the provisions of Section 59(I) of the ACT's '*Land (Planning and Environment) Act 1991*'.

CRITERION A4 *Importance for association with landmark events, developments or stages in Australian history or in the history of a state, region or community.*

Lake Burley Griffin, the lake impounded by Scrivener Dam, has a strong association with the development of Canberra as Australia's capital city. It was constructed to create a ornamental water feature on the Molonglo River which passes through the centre of Canberra and thereby covers a flood plain unsuitable for other construction purposes. It is the centre-piece of the Central National Area of Canberra and forms an important part of the immediate foreground of the Parliamentary Zone.

The idea of a lake as a central feature of Canberra emerged as a part of the 1909 proposals for the site for the national capital. The surveyor, Charles Robert Scrivener, who was instrumental in the selection of the site for Canberra gave one of the reasons for recommending the Canberra site as the opportunity afforded for 'storing water for ornamental purposes at reasonable cost'.

Although not constructed until the late 1950s - early 1960s, the lake was envisaged as a key element in the original 1913 Walter Burley Griffin town plan for Canberra. The formal central basin of the lake was placed astride the main land axis of the city with adjoining lake basins forming a 'water axis'. The lake was also considered to be part of the 'garden city' concept for Australia's capital city.

Although Canberra has achieved self government, Scrivener Dam and Lake Burley Griffin are still classed as national assets administered by the federal government.

References -
NCDC, *'Tomorrows Canberra'*, 1970
Archer K F, *'Canberra: Myths & Models'*, 1984
Reid P, *'Canberra following Griffin: a design history of Australia's National Capital'*, 1995
Holford Sir W, *'Observations on the future development of Canberra, ACT'*, 1958
NCPA, *'Lake Burley Griffin Management Plan'*, 1994

CRITERION D2 *Importance in demonstrating the principal characteristics of classes of human activities in the Australian environment (including way of life, custom, process, land-use, function, design or technique).*

Although by no means unique Scrivener Dam is a fine example of a concrete gravity dam with a large gated spillway. The gates are sized to release flood waters thereby keeping the lake level reasonably constant up to a pre-determined rate of inflow. The dam was constructed with a gated spillway due to this ability to control lake level and its ability to provide a degree of flood control.

With five bays of 30 metre long by 5.5 metre high flap gated spillway Scrivener Dam is one of the largest 'flap' gated dams in Australia. The gates themselves are one of the largest 'flap' gates in Australia *{only Meadowbank Dam in Tasmania is known to have larger 'flap' gates}*.

CRITERION E1 *Importance for a community for aesthetic characteristics held in high esteem or otherwise valued by the community.*

Lake Burley Griffin impounded by Scrivener Dam has particular aesthetic value in that its unifies the northern and southern parts of Canberra previously separated by the Molonglo River flood plain.

The lake has been sensitively constructed to complement the Canberra 'cityscape' and suitably landscaped befitting its environment and national role. Development around the lake shoreline is strictly controlled in keeping with its recreational and national significance.

The lake forms a backdrop for many of Canberra's more significant national buildings such as the National Library, National Gallery and High Court. The lake and its landscaped surrounds also forms a forecourt vista for other national buildings such old provisional Parliament House and new Parliament House.

CRITERION G1 *Importance as a place highly valued by a community for reasons of symbolic, cultural or social associations.*

Since its completion in 1963 Lake Burley Griffin has become a centre for water and land based recreation in Canberra and is highly regarded by the local community for this purpose. The lake is a venue for recreational sailing and has hosted many Australian sailing championships. It is also the base for at least two significant Australian triathalon events as well as many local similar events.

The lake shore is the venue for 'Floriade', a highly regarded floral display which attracts many visitors to Canberra. The lake backdrop is considered to be a large part of the attraction of 'Floriade'.

Reference - NCPA, '*Lake Burley Griffin Management Plan*', 1994

CRITERION H1 *Importance for close associations with individuals whose activities have been significant within the history of the nation, state or region.*

The dam and its impounded lake have a strong association with many nationally important figures in the history of Canberra as Australia's national capital. Two of the more prominent are Charles Robert Scrivener, the surveyor after whom the dam is named, and Walter Burley Griffin, the Townplanner/Architect responsible for the original plan of Canberra as Australia's capital.

In 1909 Charles Robert Scrivener recommended to the Australian parliament that Canberra be selected as the site for Australia's capital city. In doing so he suggested that an artificial lake be created by damming the Molonglo River west of the proposed site for Canberra. Scrivener's recommendations were accepted and ratified by the Australian parliament in the *'Seat of Government Acceptance Act'* of 1909. After being appointed as the first director of Commonwealth lands and surveys, Scrivener went on to be associated with many other Canberra landmarks such as the Cotter Dam, Canberra's first water supply dam. Scrivener was commemorated in 1963 when the dam forming Canberra's ornamental lake was named after him.

In 1913 Walter Burley Griffin, an American Architect, with the assistance of his wife Marion Mahony Griffin won an international competition for the townplanning of Canberra for the Federal government. Griffin's plan envisaged a chain of 5 ornamental water features along the Molonglo River flood plain which was in line with suggestions made by Scrivener in 1909. It was not until many years later that the water features were eventually built and then not to the exact plan put forward by Griffin but close to the general design envisaged. Walter Burley Griffin is commemorated in the name of the lake.

References - References as given for Criterion A4
Nairn & Serle, '*Australian Dictionary of Biography*', Vol 9, 1983
Serle, '*Australian Dictionary of Biography*', Vol 11, 1988

8. DRAWINGS AND PHOTOGRAPHS

- DRAWINGS:

  Figure 1. Location and Physical Setting (at Section 2) – (courtesy of National Capital Authority and NSW Department of Land and Water Conservation)

  Figure 2. Scrivener Dam – General Arrangement (Courtesy National Capital Authority)

  Figure 3. Walter Burley Griffin original (1912) plan for Canberra

  Figure 4. Lake Burley Griffin – showing completed lake in relation to major roads and lakeside features (courtesy National Capital Authority)

- PHOTOGRAPHS:

  A. Top - SCRIVENER DAM
  - down stream face

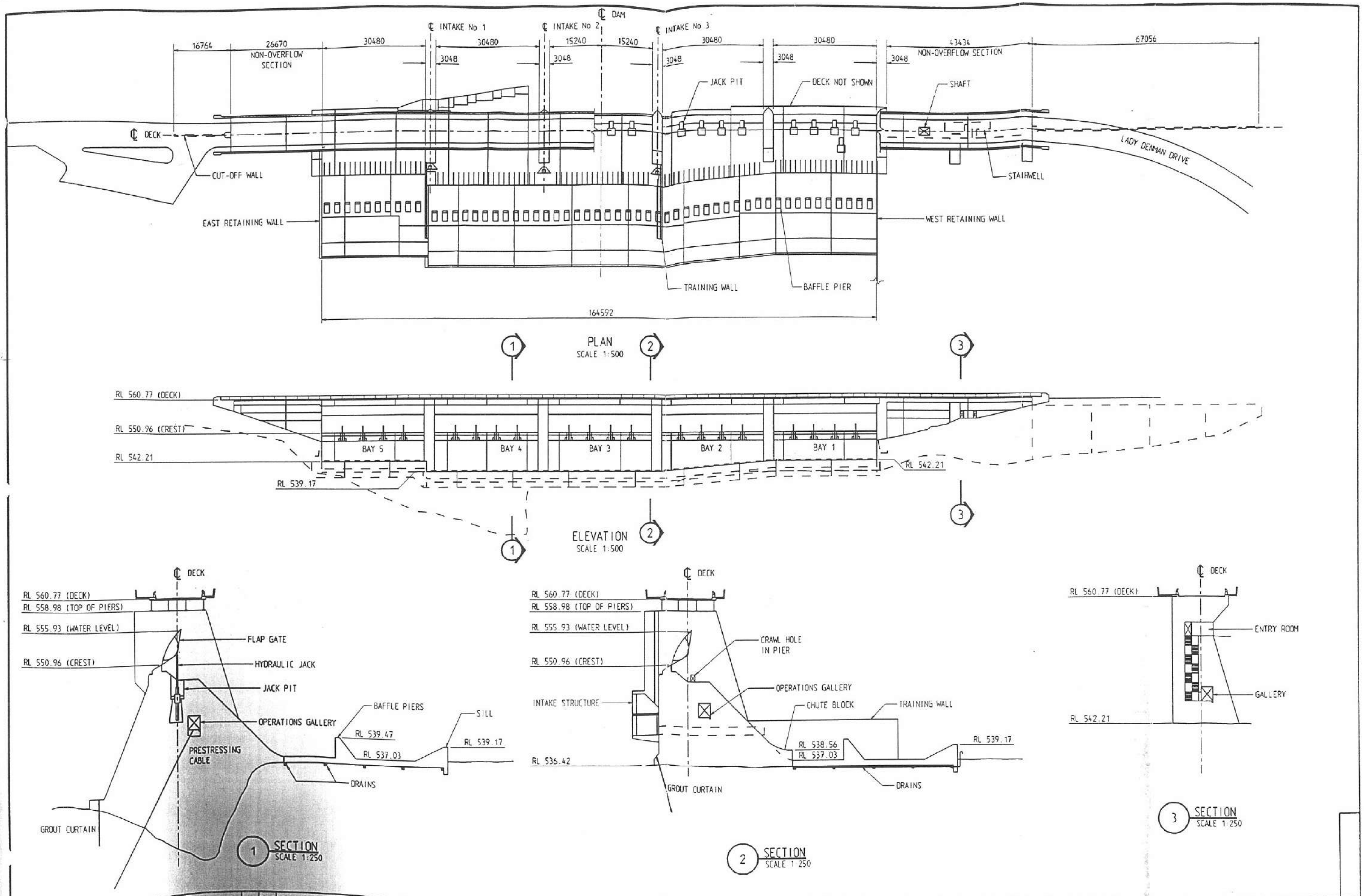
  Lower - LAKE SURFACE
  - immediately up stream from Dam

  B. Top - COMMONWEALTH AVENUE BRIDGE
  - view across Lake, to west

  Lower - COMMONWEALTH AVENUE BRIDGE
  - side view of parallel structures

  C. Top - CENTRAL BASIN OF LAKE
  - foreground to Parliamentary Zone, with bridges visible at extreme left and right sides.

  Lower - KINGS AVENUE BRIDGE AND LAKE FORESHORE



C A D D	No.	Description	Date	Init	No.	Description	Date	Init
				AMENDMENTS				

Drawn MAJW
Checked M SMITH
Design Authority M SMITH

National Capital Planning Authority
10-12 Brisbane Ave, Barton, ACT
GPO Box 373, Canberra 2601, Australia
Phone: (06) 271 2888 Fax: (06) 273 4427

SCRIVENER DAM

GENERAL ARRANGEMENT OF DAM

Scale AS SHOWN
Date JUL 96
Sheet No
File
Tender No.
CADD FILE Name GENERAL-ARRANGEMENT
Drawing No
Amend

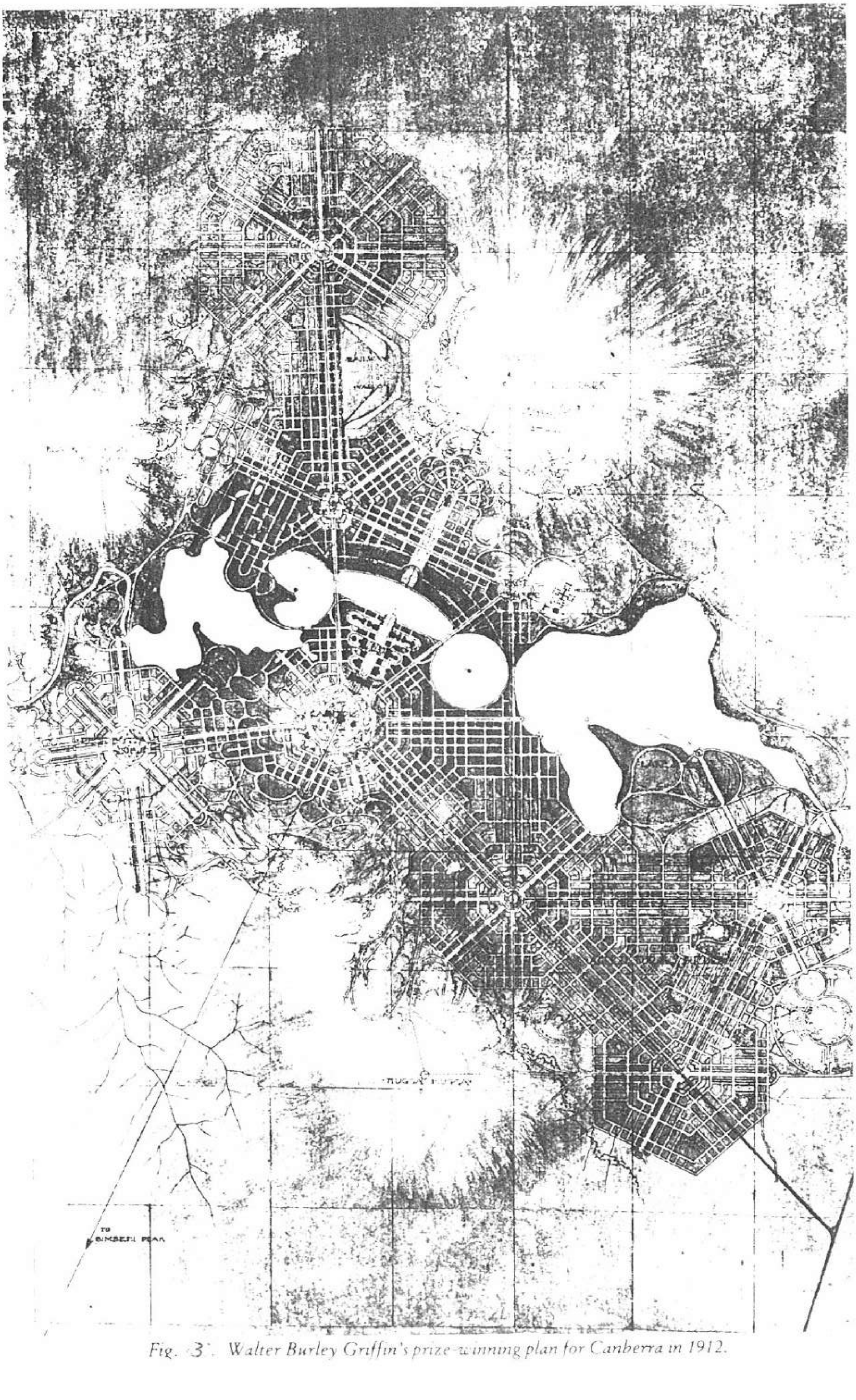


Fig. 3. Walter Burley Griffin's prize-winning plan for Canberra in 1912.

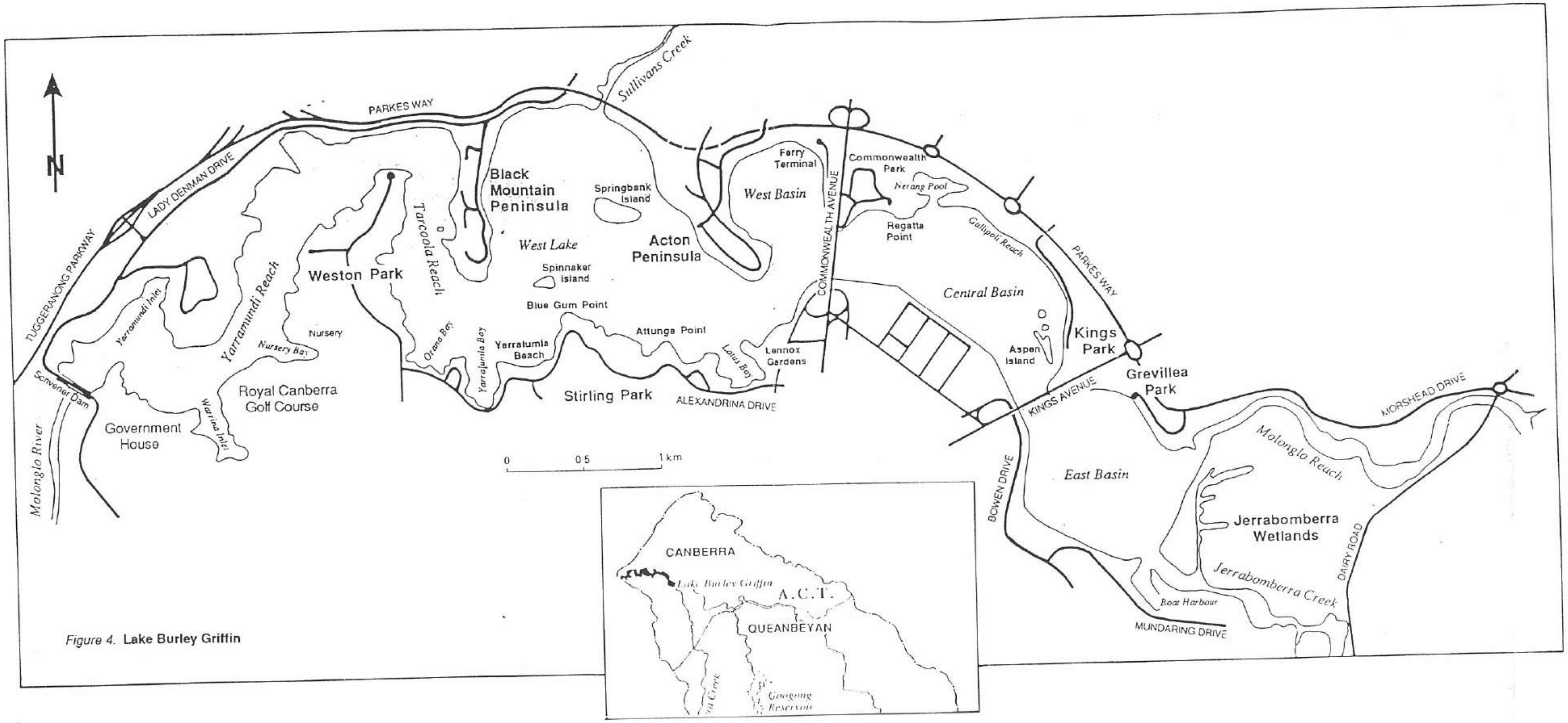


Figure 4. **Lake Burley Griffin**

A

B

C

9. CONSULTATION WITH OWNER

The National Capital Authority is the Commonwealth body responsible for management operation and maintenance of the Lake Scheme. The Authority has been advised of the intention to nominate the Scheme as a National Engineering Landmark (copy Canberra Division's letter of 28 September 2000 attached)

Officers of the Authority have assisted this submission by discussion and by making available appropriate plans, photographs and publications.

They have indicated that a formal response will be made after consideration by the Authority's relevant advisory committees.

The Institution of Engineers, Australia
CANBERRA DIVISION

28 September, 2000

Ms Annabelle Pegrum
Chief Executive
National Capital Authority
10-12 Brisbane Avenue
BARTON ACT 2600

Dear Ms ~~Pegrum~~ Annabelle,

The Institution of Engineers, Australia has a programme for public recognition of significant engineering works, and the Heritage Panel of the Institution's Canberra Division wishes to nominate Lake Burley Griffin and associated works for a "National Engineering Landmark" award under that programme. Such an award would be presented by the Institution in the form of an appropriately worded bronze plaque installed at an agreed location.

Before formal nomination to the Institution's Commemorative Plaque Sub-Committee, we seek agreement from the National Capital Authority, as the agency with jurisdiction over the Lake and associated works – in effect, the owner. Formal recognition by the Institution of the historic and engineering significance would not imply any future involvement in operation, maintenance, use or modification of the works.

A copy is enclosed of the Institution's "Guide to the Australian Historic Engineering Plaquing Programme" which describes in more detail the objectives and matters we need to address in the nomination, and arrangements for the presentation ceremony, following a successful nomination. The committee for planning this function would include representation from the Authority.

Canberra Division will host the Institution's Eleventh National Conference on Engineering Heritage on 8 and 9 October 2001. This has been endorsed as a Centenary of Federation activity by the National Council for the Centenary of Federation, and if our nomination for Lake Burley Griffin is successful we anticipate the plaque presentation ceremony will be part of the Conference programme.

Research and preparation of the nomination submission will be undertaken entirely by voluntary effort by the Canberra Division Heritage Panel. We will appreciate any support you may be prepared to offer, particularly by consultations with appropriate officers of the Authority, use of site plans and relevant documents, photographs and public relations publications.

Our contact point is Mr Byrne Kenny MIEAust (home phone 6281 5249), and the Division Director, Mrs Vesna Strika (phone 6273 1314). Mr Kenny has made preliminary contact and is available for more detailed discussion, if this would assist your consideration.

Yours sincerely

John Neal

John Neal
President, Canberra Division

PO Box E66 Kingston ACT 2604 Phone (02) 6273 1314 Fax (02) 6273 2051
ABN 70 549 545 973

10. REFERENCES

The following papers are attached as part of this nomination:

R.A. PRIDDLE "History and Development of Australia's National Capital" Presidential Address to 44th AGM of The Institution of Engineers, Australia. P.41 to 46 in The Journal of The Institution – April 1964.

A.J. CONDON, B.V. KEARSLEY and A. FOKKEMA
" Model, Spillway Model and Dam Design for the Canberra Lake Scheme"
P.195 – 205 in The Journal of The Institution of Engineers, Australia – September 1964.

E.M. BURKETT & G.N. FERNIE
"Bridges in Canberra Central Lake Area – Design"
P.139 – 150 in The Journal of The Institution – July – August 1964.

MIKE SMITH & LENORE COLTHEART
"Scrivener Dam – Nomination for Listing on the Register of the National Estate" for I.E.Aust and Australian National Committee on Large Dams – January 1998. (relevant sections only)

CLIVE J. PRICE "Lakes and Dams" – Chapter 4 in "Canberra's Engineering Heritage – Second Edition 1990.

Information from the following documents is included in the Submission:

NATIONAL CAPITAL PLANNING AUTHORITY
"Lake Burley Griffin Plan of Management Issues Paper" – February 1992.

AUSTRALIAN HERITAGE COMMISSION
"A Review of Major Dams and the Development of Dam Technology in Australia" by I.E.Aust National Committee on Engineering Heritage and Australian National Committee on Large Dams"

JOHN OVERALL "Canberra Yesterday, Today and Tomorrow – a personal memoir" – published 1995 by Federal Capital Press of Australia.

W.C. ANDREWS "Roads and Bridges" – chapter 1 in "Canberra's Engineering Heritage" Second Edition 1990.

NSW DEPARTMENT OF LAND AND WATER CONSERVATION
"The Lake Burley Griffin Catchment Protection Scheme 1965 – 1988 – A Short History" published 2000.

THE INSTITUTION OF ENGINEERS AUSTRALIA
1999 Report to Australian Heritage Commission on Heritage Dams in Australia.

DRAFT – INFORMATION PLAQUE WORDING

IEAUST. SEAL

LAKE BURLEY GRIFFIN SCHEME

First identified in 1908 as potential "ornamental water" for the new Federation's capital, the scheme was constructed in 1957-63 by National Capital Development Commission.

Formed by Scrivener Dam and bridged at Commonwealth and Kings Avenues, the lake covers 634 hectares with 33 kilometers of shoreline.

This major engineering contribution to the Australian Constitution's "seat of government" is an outstanding aesthetic setting for national buildings and public social/recreation activities.

Dedicated during Centenary of Federation 2001 by the Institution of Engineers, Australia and the National Capital Authority.